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Harry L. Englebright Lake
Yuba River, California

Seismic Evaluation of
Englebright Dam

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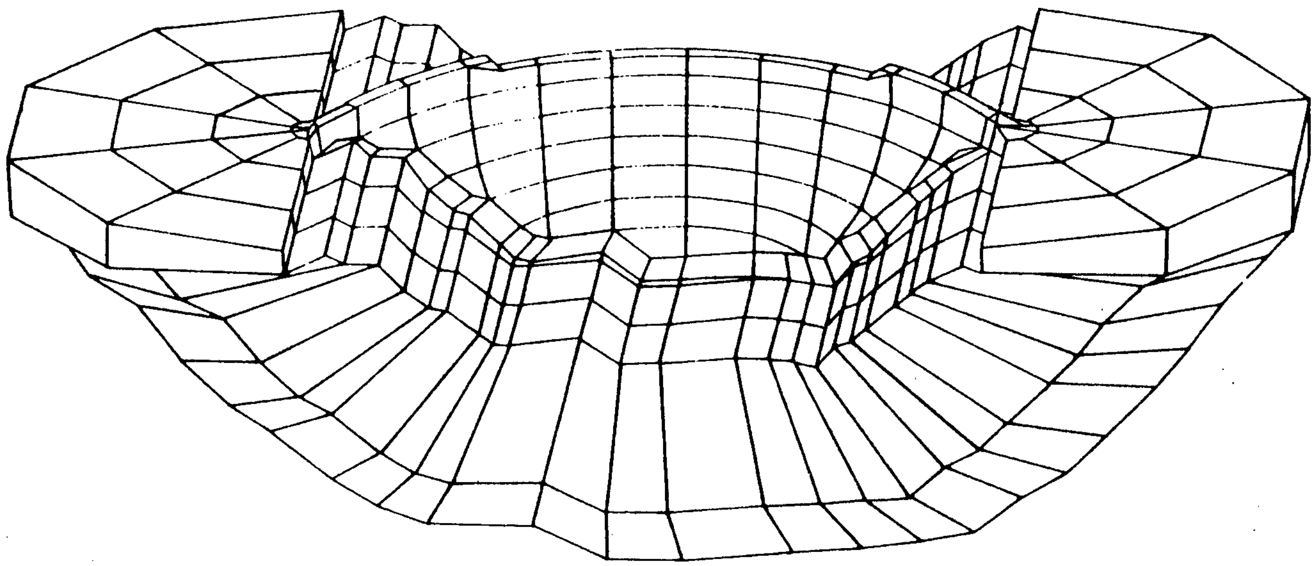

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HARRY L. ENGLEBRIGHT LAKE
YUBA RIVER, CALIFORNIA

SEISMIC EVALUATION OF ENGLEBRIGHT DAM

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CHAPTER 1

INTRODUCTION

1.1 Authority.

This report was prepared in conformance with ER 1110-2-1806, "Earthquake Design and Analysis for Corps of Engineers Projects," dated 16 May 1983.

1.2 Purpose and Scope.

The study was performed to assess the seismic safety of Harry L. Englebright Dam under maximum credible earthquakes.

1.3 Location.

Englebright Dam is located on the Yuba River about 20 miles northeast of Marysville, California. The dam is in Yuba and Nevada Counties which are part of Seismic Zone 3 (see Figure 1-1). Location of the dam is shown in Figure 1-2.

1.4 Project History.

Englebright Dam was built as part of the Sacramento River and Tributaries project to control debris from hydraulic mining operations upstream on the Yuba River. Construction began in 1939 under contract to the team of Arundel Corporation and L. E. Dixon Company. The project was completed in 1941. In 1942, a Congressional war order closed the gold mines in the state of California. The order was lifted in 1945, but mining activities were never fully resumed. In 1945, the dam's name was changed from Upper Narrows Dam to Harry L. Englebright Dam. Mr. Englebright was the Nevada Congressman who was instrumental in securing an amendment to the act which allowed construction of high dams to revive hydraulic mining. Mining activities have ceased since the dam's completion in 1941, and the reservoir is now used for hydroelectric power and recreation. The minor amount of debris deposited in the reservoir,

due to the natural movement of material from upstream areas, does not affect the use of the reservoir.

1.5 Description.

Englebright Dam is a constant-angle, concrete arch, overflow dam. The dam rises 260 feet above the streambed. The total crest length is 1,142 feet; top crest width is 21 feet. A central overflow section in the dam is depressed 15 feet with 5 bays for a total net length of 486 feet. A slight overhang at the top in the central section permits overpassing water to fall clear of the base of the dam. The plunge pool directly below the dam serves as a water cushion to dissipate the energy of the overflowing water. Figures 1-3 through 1-8 show the plan, elevation, and sectional views of the dam. Table 1-1 lists general pertinent data.

1.6 Construction History.

Englebright Dam was constructed from 1939 to 1941. Specifications for the project required completion of all excavation before placing of concrete. A further requirement was for excavation to begin at crest elevation and to proceed downward. Excavation began in the spring of 1939. River diversion was accomplished in November 1939. Streambed excavation was completed, some forms set, and the initial concrete placed by December 1939. The concrete was placed in 5-foot lifts, in alternating 50-foot long blocks. Vertical contraction joints were placed at 50-foot intervals and grouted to offset shrinkage of the concrete. Keys were provided at the joints to lock the sections together. Progress was interrupted, however, by heavy rains and subsequent flooding in December 1939 and January 1940.

From the end of January until the middle of June 1940, the contractor confined construction activities to the placing of blocks on both abutments. A slide during construction caused the placement of concrete steps on the downstream side of the right abutment. These steps spread the footing of the

dam, added strength, and aided workers in setting the forms which had been washed away or badly damaged by the winter storms.

Concreting operations resumed in June with the placement of the concrete in the streambed section. Construction of the center portion of the dam then proceeded. Final excavations were completed by the end of the summer of 1940. Concrete placing continued from the first of August until December 19, 1940 when the final lift was placed. The following day the closure gate over the diversion opening was dropped and filling of the reservoir began.

On December 21, 1940 a heavy rainstorm swept over the watershed of the Yuba River. The gate of the power outlet structure was left open, but the 70,000 acre-foot reservoir quickly filled. Englebright Dam spilled for the first time on December 26, 1940. The next day the head over the spillway reached 7.2 feet, corresponding to a discharge of 34,000 second feet. Once the storm and the Yuba River subsided, the contractor formed and poured the aeration piers on the spillway crest.

1.7 **Major Features**.

a. **Spillway**.

The initial spillway capacity, without overflowing the higher concrete section, was 110,000 cfs. Since project completion, the spillway capacity has been reduced to 102,000 cfs by modification of the spillway ends. This modification was prompted by erosion to the abutments and to the downstream face of the dam near the abutments in 1944 and 1952. The erosion was caused by water passing over the overflow section of the dam. The damaged section was cleaned and repaired with concrete in 1944 and with gunite in 1952. The overflow section was then narrowed 20 feet to prevent further erosion to the base of the dam. The section is depressed 15 feet, making its present length 486.4 feet. Four aeration piers let in air underneath the overflowing water.

b. **Instrumentation**.

Instrumentation was used to guide selection of the proper time for grouting the contraction joints. Fifty-two strain meters, 8 joint meters, and 53 resistance thermometers were installed at strategic locations during dam construction. All but 2 of the 23 monoliths in the dam contained one or more thermometers. This instrumentation provided data on stress and temperature distributions in various sections of the dam.

Results of these initial stresses and temperature distribution studies are as follows:

(1) The rate of dissipation of the heat of hydration from the concrete varied directly with the section thickness;

(2) The time lag for influence of external temperature fluctuations varied directly with the section thickness;

(3) Approximately 93 percent of the total heat had to be lost before seasonal temperature fluctuations became effective.

After approximately 99 percent of the total initial heat had dissipated, the temperature curve for the concrete interior assumed a consistent, uniform, sinusoidal pattern. The curve fluctuated about the average mean annual air and water temperature at a particular elevation.

No instrumentation data have been recorded since 1946. Observations were unnecessary due to the dissipation of the total initial heat. No new instrumentation has been installed.

c. **Hydroelectric Power Plants**.

Outlet facilities were built to allow reservoir releases for hydroelectric power generation, and two power plants have been built at the

dam. Power Plant No. 1 is owned, maintained, and operated by Pacific Gas and Electric Company. The plant uses an extension of the diversion tunnel near the right abutment. The powerhouse is an indoor type. The generator is rated 11,000 KVA. Maximum load is 11 MW. Power Plant No. 2 is owned, maintained, and operated by the Yuba County Water Agency. It is located approximately 400 feet downstream of the right abutment. The powerhouse is a single unit, outdoor type. Maximum load is 55.5 MW.

1.8 <u>Drainage Basin</u>.

The drainage basin tributary to Englebright Dam is drained by the Yuba River. The basin comprises 1,108 square miles of foothill and mountain country on the westerly slope of the Sierra Nevada. The easterly, or higher part of the basin rises to a maximum elevation of over 9,000 feet. Reservoir topography is shown in Figure 1-9.

This region experiences heavy snowfall. The maximum record snowfall at Donner Summit in the southeast corner of the drainage basin was 314 inches on 18 March 1952. The average yearly maximum snow depth in this area for the period 1941 through 1985 is 133 inches. The average annual snow depth for 1975 through 1985 is 141 inches. Thus, frequent heavy runoff has occurred at the damsite.

1.9 <u>Hydrology - Probable Maximum Flood</u>.

Reservoir control data recorded from 1973 through 1985 indicate that Englebright Dam spilled 10 of those 13 years. The total number of spills during that period was 42.

Floods on the Yuba River have been frequent, especially during the last 150 years. Extensive streamflow records were not taken prior to 1955, but since 1955 three floods were recorded with peak flows exceeding 110,000 cfs. The largest peak flows of the Yuba River at Englebright Dam are shown in Table 1-2.

The characteristics of Englebright Dam's inflow and outflow hydrographs are almost identical. The difference in peak flow is only 1,000 cfs with a peak storage of 100,000 acre-feet.

The capacity of the central overflow section of the spillway has been exceeded three times during the period 1950 through 1985. Inspection reports and work items show that these events caused significant erosion to the base of the dam. The ability of the dam to withstand long duration overflows that exceed 200,000 cfs is important to its safety. Table 1-3 lists the theoretical number of hours that the given flows will be exceeded during the Probable Maximum Flood.

1.10 Pool Elevation.

The highest pool elevation was 548.3 feet on 22 December 1964. Mean pool elevation from 1955 through 1985 was 515.69 feet. The normal operating level range is Elevation 520 to 524 feet. This level allows optimal use of the two hydroelectric power plants.

CHAPTER 2

GEOLOGY

2.1 Geologic Features.

Englebright Dam is located in the northern part of the Western Sierra Nevada Metamorphic Belt. This belt consists of a series of northwest-trending assemblages of metamorphosed Paleozoic and Mesozoic volcanic and sedimentary rocks that have been intruded, primarily along the east side, by plutonic rocks of the extensive Mesozoic Sierra Nevada batholith. The structure of the metamorphic rocks is complex because folding, faulting, and intrusion have caused successive deformations. The major geologic structures in the area strike about N. 25° W. and dip steeply northeast.

Englebright Dam is underlain by metavolcanic rocks, probably of Jurassic age. These rocks were intruded by plutonic rocks of Jurassic to Cretaceous age. The metavolcanic rocks originally consisted of volcanic tuffs, breccias, and flows with some pillow-structured basalts which indicate the material was extruded into water. The plutonic rocks range in composition from basic intrusives to acidic quartz diorite and granodiorite.

The entire bedrock complex at Englebright Dam consists largely of dikes, interdike metavolcanic rocks, and intrusive quartz diorite. The river section, lower canyon slopes, and much of the right abutment are predominantly quartz diorite which is a light-colored, fine to medium grained, massive, crystalline rock. This quartz diorite is cut by scattered uralite diabase and meta-basalt dikes. The left abutment and part of the upper right abutment are predominantly metavolcanic rocks. These rocks consist of uralite diabase, meta-andesite, and meta-andesite tuff breccia.

The uralite diabase underlies a considerable portion of the downstream foundation area. It is a dark-colored, fine grained, massive, crystalline rock that is partly intrusive and partly indicative of massive lava flows. The meta-andesite is found in both abutments and consists of metamorphosed

lava flows. The meta-andesite tuff breccia was originally a volcanic lapilli tuff or tuff breccia, the product of explosive eruptions. This breccia was consolidated by compression and metamorphism into massive rock.

All of the foundation rocks are moderately to highly fractured and jointed. They contain numerous tight shears and fault zones. Several narrow fault zones were described in the foundation explorations; some were encountered during foundation excavation. The most prominent fault traverses the upper left abutment and was encountered and explored in the lower left abutment. It also extends a few hundred feet downstream from the dam on the right side of the river. The fault strikes about N. 74° E. and dips 73° to 80° northwest.

2.2 **Foundation Explorations**.

Foundation explorations consisted of eighteen diamond core drill holes; three tunnels in the right abutment; two tunnels in the left abutment; a few inclined shafts; and some trenches. Drill logs and data are available only for Drill Holes 12 through 18 in the canyon bottom. No logs or detailed descriptions are available for the other explorations. A brief summary of some data is available in an undated "Geological Report on the Upper Narrows Damsite," by Consulting Geologist George D. Louderback. The probable date of the report is 1938. The document refers to a geologic map, geologic profile, logs of drill holes, and other geological interpretation data.

2.3 **Foundation Excavation**.

No data are available on foundation excavation. However, since contract specifications required completion of all excavation before placing concrete, excavation is assumed to have been done in that manner, beginning at the crest and proceeding down the abutments. Initial excavation was probably done by dozers, followed by drilling and blasting. Faulted and fractured rock sloping riverward and downstream was encountered during excavation. This discovery resulted in substantial redesign of the dam, and the quantity of excavation for that area was doubled.

Most excavated materials were removed by heavy equipment; final mucking and cleanup were done by hand labor. The total amount of excavated material is unknown. All rock surfaces were cleaned by washing with high pressure air-water hoses to prepare for concrete placement.

2.4 Foundation Treatment.

As in the cases of foundation exploration and excavation, very little data are available on foundation treatment. According to standard practice of the time, slurry grout was probably placed on the exposed foundation to fill fractures and depressions. Tunnels in the abutments were probably backfilled with concrete.

A slide occurred on the right abutment during construction of Englebright Dam. Consequently, a series of concrete steps was constructed up the right abutment in 5-foot lifts, with vertical contraction joints at 50-foot intervals. These steps added strength to the dam and aided in setting the forms.

Foundation drilling and grouting were performed although no data or drawings are available. A single-line grout curtain was probably constructed. Grout pipes were installed in the foundation rock prior to initial concrete placement.

CHAPTER 3

SEISMICITY AND ACCELEROGRAMS

3.1 Seismicity and Seismic Intensity.

A report entitled "Seismicity and Seismic Intensity Study, Englebright Dam, California," dated January 1983, by Bruce A. Bolt, and included as Appendix A, was used to determine the seismic parameters of design earthquakes which might be expected in the vicinity of Englebright Dam.

The report concludes that a maximum earthquake would occur from 6 to 10 km away from the dam and that its magnitude would be from 6.25 to 6.5 on the Richter scale. The report stresses, however, "... that an earthquake of this magnitude is very unlikely within the lifetime of the dam and its adoption would provide an upper bound to feasible ground motions for engineering analysis."

3.2 Selection of Accelerograms.

Based on Dr. Bolt's report, a second report entitled "Selection of Accelerograms for Seismic Safety Evaluation of Englebright Dam, California" by H. Bolton Seed, undated and included as Appendix B, provided two accelerograms for use in engineering studies of Englebright Dam. These accelerograms are designated as follows:

M6.5-8K-83	Magnitude	=	6.5	Richter
	Peak acceleration	=	0.5	g
	Peak velocity	=	37.5	cm/sec
	Duration	=	18.0	sec
	Time step	=	0.01	sec
M6.4-8K-83	Magnitude	=	6.4	Richter
	Peak acceleration	=	0.5	g
	Peak velocity	=	41.5	cm/sec
	Duration	=	16.0	sec
	Time step	=	0.02	sec

3.3 <u>Modification of Accelerograms</u>.

The accelerograms provided by Dr. Seed represented earthquake shaking in a direction normal to the axis of the dam. Since ground motions during an earthquake are more appropriately described by three components of shaking, it was necessary to modify the accelerograms to include motions in three directions:

1. Normal to the axis of the dam
2. Along the axis of the dam
3. Vertical motions

In a letter dated July 12, 1985 and included as Appendix C, Drs. Bolt and Seed recommended that the two additional components be generated as follows:

M6.5-8K-83

1. Motions along the axis of the dam should be the same as motions normal to the axis but with a phase shift of 0.45 seconds.

2. Vertical motions should be the same as motions normal to the axis but with the frequency increased by a factor of 1.5 and the amplitudes reduced by a factor of 0.5.

M6.4-8K-83

1. Motions along the axis of the dam should be the same as motions normal to the axis but with a phase shift of 0.50 seconds.

2. Vertical motions should be the same as motions normal to the axis but with the frequency increased by a factor of 1.5 and the amplitudes reduced by a factor of 0.5.

Due to their increased frequency, the resulting vertical components had time steps (delta T) that were smaller than the time steps of the other two components. Since the analysis required that all three components have the same time step, modification of the resulting vertical components was necessary. The components were modified by deleting "extra" points while preserving local peaks.

In the following chapters, the term QUAKE 1 refers to the three components of M6.5-8k-83, and QUAKE 2 refers to the three components of M6.4-8k-83. QUAKE 1 and QUAKE 2 are maximum credible earthquake events.

3.4 Further Modifications.

In order to minimize the required calculations and optimize computer time, it was recommended by Professor Anil K. Chopra of the University of California at Berkeley that accelerogram M6.5-8K-83 be modified to have a time step of 0.02 seconds instead of 0.01 seconds. The accelerogram was modified by deleting "extra" points while preserving local peaks. Accelerograms and response spectra before and after modification compared favorably and are shown in Figures 3-1 through 3-4.

Figures 3-5 through 3-7 show the three components of M6.5-8K-83 and Figures 3-8 through 3-10 show the spectral accelerations for each component for two damping ratios.

Figures 3-11 through 3-13 show the three components of M6.4-8K-83 and Figures 3-14 through 3-16 show the spectral accelerations for each component for two damping ratios.

CHAPTER 4

CONCRETE AND FOUNDATION PROPERTIES

4.1 Concrete Properties.

The Corps of Engineers conducted static testing of 6 inch x 12 inch concrete cores drilled from Englebright Dam to determine the compressive strength, splitting tensile strength, modulus of elasticity, and Poisson's ratio. Rapid loading tests for these same properties were performed by the U.S. Bureau of Reclamation (USBR) Laboratory in Denver on 6 inch x 12 inch cores. Test data from the two test programs and the location maps of the concrete cores are presented in Appendix D. The USBR's tests indicated little or no increase in concrete material properties for rapid loading conditions (see Table D-5 in Appendix D). Concrete properties used in the finite element model are as follows:

Modulus of Elasticity $E_c = 4.75 \times 10^6$ psi

Poisson's Ratio $\nu_c = 0.20$

Unit Weight $\gamma_c = 155$ pcf

The above value of E_c for the dam is the arithmetic average of the static values determined by laboratory analysis of 13 test specimens taken from the dam. The value of ν_c used is the average of the dynamic results of 19 test specimens.

At the time this report was prepared, a supplemental testing program was being conducted by Professor Jerome M. Raphael at the University of California at Berkeley. The following test results were obtained from Professor Raphael prior to the release of his forthcoming report entitled "Mass Concrete Tests - Englebright, Folsom, Pine Flat."

Average Static Splitting Tensile Strength = 466 psi

Average Dynamic Splitting Tensile Strength = 624 psi

check these!

The above values are adopted as a measure of the tensile strength of the dam concrete.

4.2 Foundation Rock Properties.

The Corps of Engineers conducted testing of rock cores taken from the Englebright damsite. Test results and the location map of rock core drill sites are included in Appendix D. Foundation rock properties used in the finite element model are as follows:

Modulus of Elasticity $E_f = 12.48 \times 10^6$ psi

Poisson's Ratio $\nu_f = 0.24$

The above values of E_f and ν_f for the foundation rock are arithmetic averages of the values determined by laboratory testing of 12 rock core samples taken from the damsite.

CHAPTER 5

DESCRIPTION OF COMPUTER PROGRAMS

5.1 GTSTRUDL and EACD.

A three-dimensional finite element analysis of Englebright Dam was accomplished with the use of two computer programs: GTSTRUDL and EACD.

GTSTRUDL is a general purpose, structural engineering software program with finite element capabilities. The program is a product of the GTICES Systems Laboratory of the School of Civil Engineering at Georgia Institute of Technology. GTSTRUDL Version 84.03 is used in this study. The program runs on a CDC CYBER 176 computer with 377K words of central core memory and 1000K words of extended core memory.

EACD is a computer program designed specifically for three-dimensional static and dynamic analyses of concrete dams. The program was originally developed at the University of California at Berkeley by Dr. John F. Hall and was significantly modified by Dr. Ka-Lun Fok under the technical supervision of Professor Anil K. Chopra. The program was coded in FORTRAN IV for the CDC 7600 computer. The research investigation {1, 2, 3, 4} was supported by the National Science Foundation. A detailed description of input data for the program is available in reference {5}. In the study of Englebright Dam, the program operates, with minor modification, on the CDC CYBER 205 computer which has 36,400K words of central core memory and no extended core memory.

5.2 Load Cases Analyzed.

The program EACD has some limitations. It is incapable of analyzing the dam for (1) silt loading, and (2) surcharge loading (water overflowing the dam) corresponding to the occurrence of the probable maximum flood (PMF). The latter condition is commonly referred to as the unusual loading. The limitations are overcome by employing GTSTRUDL to handle the aforementioned

load cases. For the static and dynamic load cases corresponding to gross pool elevation, analysis of the dam was accomplished by EACD.

5.3 Stress Results.

As will be seen in Chapter 6, solid elements are used for discretizing a three-dimensional dam-foundation system in both GTSTRUDL and EACD analyses. Utilizing this type of elements in GTSTRUDL, stresses are output at the nodal points with respect to the global coordinate system. Thus a postprocessor was developed to convert the global stresses to the more meaningful stress quantities, namely, arch stress, cantilever stress, and principal stress.

In the case of EACD analysis, stresses are output at the Gauss Quadrature points (of order 2) as well as at the nodal points of the solid dam elements, with reference to local stress axes 1, 2, 3 which are defined at each stress location in terms of the mapped local element axes. The 1, 2, 3 axes can be oriented in the arch, cantilever, and normal directions, respectively, by properly ordering the nodes for the element connectivity {5}. Thus the arch, cantilever, and principal stresses can be computed directly from EACD. Note that eight Gauss Quadrature points are located inside each solid dam element. The points are defined with respect to the local element axes, and are numbered locally 1 to 8 for each dam element {5}.

CHAPTER 6

DAM, FOUNDATION ROCK, AND RESERVOIR FINITE ELEMENT MODELS

6.1 Dam Finite Element Models.

Four different finite element meshes are considered possible candidates for modeling the arch dam. The models are described as follows:

a. 1-Layer-8-Node Solid Finite Element Dam Model.

This dam mesh is composed of one layer of finite elements through the thickness of the dam. The finite element idealization of the dam consists of 63 eight-node solid elements and 2 six-node prismatic solid elements. The model is shown in Figure 6-1 and is abbreviated as 1L8ND model in this report. There are 174 nodes in the model which, if complete constraints are allowed for at the dam-foundation interface (58 fixed nodes), provide 348 degrees of freedom. Note that 3 translational degrees of freedom are associated with each free node in this model.

b. 1-Layer-20-Node Solid Finite Element Dam Model.

This dam mesh is composed of one layer of finite elements through the thickness of the dam. The finite element idealization of the dam consists of 55 twenty-node solid elements and 2 six-node prismatic solid elements. The model is shown in Figure 6-2 and is abbreviated as 1L20ND model in this report. There are 487 nodes in the model which, if complete constraints are allowed for at the dam-foundation interface (133 fixed nodes), provide 1062 degrees of freedom. Note that 3 translational degrees of freedom are associated with each free node in this model.

c. 2-Layer-20-Node Solid Finite Element Dam Model.

This dam mesh is composed of two layers of finite elements through the thickness of the dam. The finite element idealization of the dam consists

of 110 twenty-node solid elements and 4 six-node prismatic solid elements. The model is shown in Figure 6-3 and is abbreviated as 2L20ND model in this report. There are 768 nodes in the model which, if complete constraints are allowed for at the dam-foundation interface (213 fixed nodes), provide 1665 degrees of freedom. Note that 3 translational degrees of freedom are associated with each free node in this model.

d. **8-Node Thick Shell Finite Element Dam Model**.

This dam mesh is composed of one layer of finite elements through the thickness of the dam. The finite element idealization of the dam consists of 55 eight-node shell elements and 2 three-node prismatic shell elements. The model is shown in Figure 6-4 and is abbreviated as 8NTSD model in this report. There are 206 nodes in the model which, if complete constraints are allowed for at the dam-foundation interface (53 fixed nodes), provide 765 degrees of freedom. It is important to note that all nodes in this thick shell model are located at the mid-surface of the dam and that 5 degrees of freedom are associated with each free node: 3 translations and 2 rotations.

6.2 Determination of a Suitable Finite Element Dam Model.

A study was conducted to determine a suitable finite element idealization for Englebright Dam. To this end, the natural frequencies (eigenvalues) and mode shapes (eigenvectors) of the first 10 modes of vibration of the dam on rigid foundation rock without water were computed for each of the finite element dam models described in the preceding section. The computer program EACD was employed to perform the eigenvalue analysis (computation of natural frequencies and mode shapes) for the 1L20ND and 8NTSD models using the consistent mass approach (EACD does not have the lumped mass option). GTSTRUDL was utilized to do the same analyses for the 2L20ND model using only the consistent mass approach and for the 1L8ND and 1L20ND models using both the lumped mass and consistent mass approaches. It should be noted that the consistent mass technique is less computationally efficient but more accurate than the lumped mass approach. The computed natural frequencies for the four

idealizations of Englebright Dam are listed in Table 6-1. Features of the four finite element dam models are summarized in Table 6-2.

Comparison between the four dam models was made on the basis of the modal frequencies. The comparison reveals a close resemblance in modal frequencies only between the 1L20ND and the 2L20ND models. In addition, the mode shapes for these two models are similar (see Figures 6-5 through 6-24). This resemblance in modal characteristics between the 1L20ND and the 2L20ND models indicates that both finite element models are suitable for idealization of the dam. Since analysis using the 1L20ND model requires less computational effort than that using the 2L20ND model, the 1L20ND model is selected as the dam model used in subsequent EACD and GTSTRUDL analyses in this study. Figures 6-25 through 6-41 show the elevation and sectional views of the 1L20ND model. Node numbers and element numbers are presented in these figures.

6.3 Foundation-Rock Finite Element Model.

The flexibility effects of the foundation rock are included in EACD's analytical procedure by modeling a certain volume of the foundation rock under the dam. Theoretically, the foundation rock can be modeled to resemble the actual topography of the foundation rock at the damsite.

In the finite element analysis of Englebright Dam, the shape of the foundation-rock model is idealized using a procedure that has been adopted in the computer program ADAP {6}. The foundation mesh generated by this procedure is believed to be adequate for most practical purposes. Utilizing the above-mentioned procedure, the dam canyon is assumed to be prismatic in the upstream direction, as shown in Figure 6-42. check w/ White.

Basically, the foundation mesh can be thought of being defined by a group of inclined semicircular planes. The nodal point arrangement on a typical semicircular plane is shown in Figure 6-43. Foundation nodes 1 and 3 are respectively in contact with the downstream abutment nodal point and the upstream abutment nodal point on the faces of the dam. Nodes 1 and 3 usually

do not lie on the semicircular plane. Foundation node 2 corresponds to the dam abutment mid-surface node, midway between the corresponding pair of upstream and downstream abutment nodal points. Node 2 is the common center of the three semicircles drawn in dashed lines. It is apparent that foundation nodes 1, 2, 3, 4, 5, 6, 7, 8, 9 constitute an irregular surface (shaded portion) joined together in space with the semicircular plane (unshaded portion). Note that node numbers assigned to the foundation nodal points in Figure 6-43 are intended to show the order of node numbering in the foundation mesh.

As can be seen from Figure 6-43, there is a total of 18 quadrilaterals on each of the inclined semicircular planes (including the irregular surface mentioned above). Thus 18 three-dimensional solid elements are formed between two consecutive inclined planes. The radius R_f of the semicircular plane, therefore, controls the total volume of the foundation-rock region included in the analysis. The radius parameter R_f should be chosen to be large enough to satisfactorily represent foundation flexibility effects in the response analysis of the dam. It was found that the ratio E_f/E_c provides a basis for selection of a value of R_f {3}, where E_f and E_c are respectively the Young's modulus of the foundation rock and the dam concrete.

For the case of Englebright Dam with $E_f/E_c = 2.6$ (see Chapter 4), $R_f = 1.5H_d$ is selected to adequately represent the foundation-rock flexibility effects, where $H_d = 267$ feet is the height of the dam. Having selected a value for R_f, a foundation mesh is constructed for the 1L2OND dam model using a mesh generation computer program already developed to implement the above-mentioned procedure. With a total of 27 semicircular planes defined at the dam abutment mid-surface nodes along the dam boundary, a foundation mesh is formed. Figure 6-44 shows the foundation mesh constructed for the 1L2OND dam model, and Figure 6-45 depicts the foundation mesh supporting the dam mesh.

The three-dimensional finite element idealization of the foundation-rock region, shown in Figure 6-44, consists of 468 solid finite elements with 781 nodal points. Of the 468 solid elements, 26 twelve-node elements and 52 nine-node elements are modeled in order for the dam mesh and foundation mesh geometrically match each other along the dam-foundation interface. The rest of the 468 elements are modeled as eight-node elements. This foundation mesh will be adopted in subsequent analyses of Englebright Dam using the program EACD (EACD permits use of variable 8 to 20 node solid elements).

In analyzing the dam using GTSTRUDL for the cases of surcharge loading and silt loading, slight modification of the above-described foundation mesh is necessary because GTSTRUDL does not have nine-node solid element and twelve-node solid element in its element library (GTSTRUDL permits use of either eight-node or twenty-node solid elements). It is therefore evident that the foundation mesh should be modified by omitting all the non-corner nodes of the nine-node elements and the twelve-node elements. The mesh, after modification, consists of 468 eight-node solid elements and will be used in GTSTRUDL analysis of the dam. It should be noted that the modification causes minor violation of geometric compatibility at the dam-foundation interface.

6.4 Boundary Conditions.

When foundation-rock flexibility is considered, the external boundaries of the foundation mesh are fixed in the finite element analysis, i.e., nodal points at the outer rims of the semicircular planes of the foundation mesh are fixed. In this case, 162 foundation nodal points are fixed. Note that each free foundation nodal point has 3 degrees of freedom: X, Y, Z translations.

When foundation-rock flexibility is not considered (completely rigid foundation rock is assumed), the dam nodal points are fixed at the dam-foundation interface. In this case, 133 dam nodal points are fixed. Note that each free dam nodal point has 3 degrees of freedom: X, Y, Z translations.

Table 6-3a summarizes the important features of the dam mesh and the foundation mesh used in EACD analysis. When analyzing the dam (including the foundation rock) using GTSTRUDL, the dam mesh and the foundation mesh are actually treated as a single finite element model. The features of this model are presented in Table 6-3b.

6.5 Material Properties.

The foundation rock and the dam concrete are assumed to be homogeneous, isotropic, and linearly elastic. The following properties are used in this study for the dam and foundation models (see Chapter 4).

Concrete:	Young's Modulus of Elasticity $E_c = 4.75 \times 10^6$ psi
	Poisson's Ratio $\nu_c = 0.20$
	Unit Weight $\gamma_c = 155$ pcf
Foundation Rock:	Young's Modulus of Elasticity $E_f = 12.48 \times 10^6$ psi
	Poisson's Ratio $\nu_f = 0.24$

Note that the foundation rock is assumed to be massless in this investigation. Only the static foundation flexibility effects are considered; the inertial and damping effects of the foundation rock are ignored in considering dam-foundation interaction effects.

6.6 Reservoir Finite Element Model.

The reservoir finite element model discussed in this section is used only in the EACD analysis. The model is not applicable to GTSTRUDL.

The reservoir behind a dam is generally of complicated shape, as dictated by the natural topography of the damsite. In the case of Englebright Dam, the reservoir extends a large distance in the upstream direction. Theoretically, finite element idealization can be used to properly model the complicated geometry of the entire reservoir. Such an idealization is considered

impractical because it would render modeling of the reservoir intractable, and would require prohibitive computational effort.

For most practical purposes, an efficient approach {1} is commonly adopted to idealize the reservoir which extends to large distance in the upstream direction, as shown in Figure 6-46. In this approach, the finite element idealization of the reservoir consists of an irregular region of finite size connected to a channel of uniform cross section extending to infinity in the upstream direction. The plane which connects the irregular region of the reservoir with the infinite channel is referred to as the transmitting plane. The irregular region is idealized as an assemblage of three-dimensional fluid finite elements. For the infinite channel, a finite element discretization of the cross section combined with a continuum representation in the infinite direction provide for the proper transmission of pressure waves. The discretization of the cross section should be compatible with that of the irregular region over the transmitting plane. This approach is used in this study to model the reservoir of Englebright Dam.

Five different finite element fluid meshes are essential to completely define a reservoir finite element model. These meshes are referred to as Mesh 1, Mesh 2, Mesh 3, Mesh 4, and Mesh 5 in this report. The definition of each mesh is presented below.

Mesh 1 spans the entire irregular region of the reservoir, with the mesh compatible with that of the dam at its upstream face. Mesh 2 spans the transmitting plane connecting the irregular region with the infinite channel. Mesh 3 spans the surface of the reservoir that touches the dam. Mesh 4 spans the bottom and sides of the reservoir in the irregular region. Mesh 5 spans the bottom and sides of the transmitting plane. Note that Meshes 2, 3, 4, 5 can be directly derived from Mesh 1.

In modeling the reservoir behind Englebright Dam, the transmitting plane is located 30 feet from the upstream base of the crown cantilever, measured in the upstream direction (X direction). The shape of Mesh 1 was modeled to

resemble, only to a certain extent, the actual topography of the reservoir. Geometric compatibility must be satisfied at the dam-water interface but minor violations of compatibility at the foundation-water interface are acceptable.

The reservoir finite element model is shown in Figure 6-47. Meshes 1 through 5 for the model are described as follows:

a. Mesh 1 is composed of 102 twenty-node finite elements, 6 seventeen-node elements, and 2 nineteen-node elements. The elements are three-dimensional. The seventeen-node elements and nineteen-node elements are modeled in order for Mesh 1 to be compatible with the dam mesh at the upstream face of the dam.

b. Mesh 2 is composed of 55 eight-node elements. The elements are two-dimensional.

c. Mesh 3 is composed of 47 eight-node elements, 6 five-node elements, and 2 seven-node elements. The elements are two-dimensional. The five-node elements and seven-node elements are modeled to satisfy compatibility between Mesh 3 and the dam mesh.

d. Mesh 4 is composed of 50 eight-node elements and 2 seven-node elements. The elements are two-dimensional. The seven-node elements are modeled to satisfy compatibility between Mesh 4 and the dam mesh.

e. Mesh 5 is composed of 26 three-node line elements. The elements are one-dimensional.

Different types of fluid (water) finite elements used in Meshes 1 through 5 are shown in Figure 6-48. These elements are derived from solutions of the pressure wave equation {1}.

The finite element idealization of the reservoir in this study consists of 748 nodal points with 95 free surface nodal points on top of the reservoir

water. Thus the idealization has 653 pressure degrees of freedom. Note that each fluid node which does not lie on top of the reservoir has one degree of freedom, i.e., hydrodynamic pressure at the node.

CHAPTER 7

MODAL CHARACTERISTICS OF THE DAM-FOUNDATION SYSTEM

7.1 Frequencies.

Natural frequencies for the first 15 modes of vibration of the dam on flexible foundation rock with empty reservoir were computed using EACD and are shown in Table 7-1. Also presented in this table are the modal frequencies of the dam with empty reservoir, with the foundation rock assumed to be rigid. A comparison study indicates that the natural frequencies of individual vibration modes of the dam generally decrease when the flexibility effects of the foundation rock are considered, i.e., the dam-foundation interaction lengthens the periods of vibration. EACD is unable to directly compute the resonant frequencies of the dam including dam-water interaction. These frequencies, which are known to be affected by dam-water interaction, have been investigated in depth in Dr. Fok's research study {3} and will not be re-investigated here.

7.2 Mode Shapes.

Mode shapes of the crown cantilever are shown in Figures 7-1 through 7-15 for the first 15 vibration modes of the dam on flexible foundation rock with empty reservoir. The corresponding mode shapes of the dam crest are depicted in Figures 7-16 through 7-30. Comparison of this set of figures with the mode-shape plots for the case of rigid foundation (see Figures 6-5 through 6-24) shows that modifications in the mode shapes due to foundation flexibility are very minor. This observation is expected because the foundation rock is much stiffer than the dam concrete (foundation-rock modulus is 2.6 times the dam-concrete modulus).

7.3 Number of Vibration Modes N_m.

To produce accurate earthquake response of the dam, all the dam's vibration modes that significantly contribute to the dynamic response should

be included. The number of dam's vibration modes necessary for accurate response was determined from a study in which the dynamic stress response of the dam was computed using N_m = 5, 10, 15, 20, 25, and 30 for each of the two earthquake excitations: QUAKE 1 and QUAKE 2. In this investigation, the dam is assumed to be supported on rigid foundation rock with an empty reservoir; and a constant hysteretic damping factor η = 0.10 for the dam concrete was selected. This damping factor corresponds to a viscous damping ratio of 5% in all natural vibration modes of the dam on rigid foundation rock with empty reservoir (see Section 9-1 of Chapter 9 for more discussion about η).

The time history of dynamic arch stress at stress location 4 in dam element 9 due to QUAKE 1 excitation for N_m = 15 and 20 is shown in Figure 7-31. The time history of dynamic cantilever stress at stress location 3 in dam element 51 due to QUAKE 2 excitation for N_m = 15 and 20 is shown in Figure 7-32. Stresses due to static loading (dead weight of the dam) are not included in the above stress responses. Comparing the stress histories in Figures 7-31 and 7-32, the close similarity between the stress response for N_m = 15 and that for N_m = 20 reveals that 15 vibration modes are sufficient for accurate evaluation of the dam response. It is therefore concluded that 15 vibration modes should be included in subsequent dynamic analysis of the dam for QUAKE 1 or QUAKE 2 excitations.

Figure 7-33 presents a plot of the peak dynamic compressive arch stress at stress location 4 in dam element 9 (due to QUAKE 1) for selected numbers of vibration modes. Figure 7-34 shows the corresponding plot for the peak dynamic tensile cantilever stress at stress location 3 in dam element 51 (due to QUAKE 2). These two figures further substantiate the above conclusion.

CHAPTER 8

STATIC RESPONSE OF ENGLEBRIGHT DAM

8.1 Static Load Cases.

The static response of Englebright Dam was analyzed for the following loading conditions:

a. Usual loading - Self weight of the dam plus hydrostatic pressure due to gross pool elevation (spillway crest elevation) of 527 feet. The dam response due to usual loading was computed using the program EACD.

b. Unusual loading (surcharge loading) - Self weight of the dam, hydrostatic pressure due to PMF pool elevation of 557.7 feet, tailwater pressure on the downstream face, and aeration pressure on the downstream face of the dam. The aeration pressure is due to water flowing over the spillway crest (EL. 527 feet). This causes a pulling effect on the downstream face of the dam. The dam response due to this unusual loading was computed using GTSTRUDL.

The stresses induced by silt load were evaluated using GTSTRUDL. In this investigation it is assumed that the pressure exerted by saturated silt is equivalent to that of a fluid weighing 85 pcf. The analysis indicates that the influence of the sediments on the static stresses in the dam is negligibly small. Thus the static effects of silt load are not included in subsequent analyses. Figure 8-1 shows the depth of silt against the upstream face of the dam.

8.2 Stress Response Due to Usual Loading.

The stress response due to usual loading is computed at the dam nodal points for the dam supported on flexible foundation rock and for the dam supported on rigid foundation rock. Table 8-1 summarizes the tensile stresses at the most-stressed locations of the dam for both cases. It can be seen from

this table that the effects of foundation flexibility significantly reduce the stress response of the dam. Considering foundation-rock flexibility, the maximum tensile stress is 207 psi at node 472 on the upstream face, 15 psi at node 55 on the downstream face, and 20 psi at node 214 on the mid-surface of the dam. Also, the maximum compressive stress is 345 psi at node 303 on the upstream face, 658 psi at node 191 on the downstream face, and 307 psi at node 217 on the mid-surface of the dam. The maximum radial displacement is 0.37 inch at dam crest nodal point 303 in the downstream direction.

The arch and cantilever stresses on the upstream and downstream faces along section 6 and section 7 (crown cantilever) are shown in Figures 8-2 and 8-3, respectively. Principal stress vectors plotted on the upstream face are shown in Figure 8-4, and those on the downstream face are shown in Figure 8-5. As can be seen from these plots, arch stresses are generally dominant near the dam crest; whereas the cantilever stresses are dominant near the base. Figures 8-6 through 8-8 are stress-contour plots presenting the distribution of arch, cantilever, and principal stresses on the upstream and downstream faces.

Stresses are also computed at Gauss Quadrature points inside the 3-D solid dam elements for the dam supported on flexible foundation rock. The maximum tensile stress is 20 psi at stress location 2 in dam element 24, and the maximum compressive stress is 411 psi at stress location 6 in dam element 50.

8.3 <u>Stress Response Due to Surcharge Loading</u>.

The stress response due to surcharge loading is computed at the dam nodal points for the dam supported on flexible foundation rock. The maximum tensile stress is 586 psi at node 479 on the upstream face, 314 psi at node 13 on the downstream face, and 511 psi at node 272 on the mid-surface of the dam. Also, the maximum compressive stress is 219 psi at node 460 on the upstream face, 752 psi at node 180 on the downstream face, and 326 psi at node 233 on the mid-surface of the dam. The maximum radial displacement is 0.914 inch at dam crest nodal point 303 in the downstream direction.

Principal stress vectors plotted on the upstream face are shown in Figure 8-9, and those on the downstream face are shown in Figure 8-10. Figures 8-11 through 8-13 are stress-contour plots presenting the distribution of arch, cantilever, and principal stresses on the upstream and downstream faces.

CHAPTER 9

EARTHQUAKE RESPONSE OF ENGLEBRIGHT DAM

9.1 Constant Hysteretic Damping Factor.

Energy dissipation in the dam concrete is represented by constant hysteretic damping with a damping factor η. A viscous damping ratio ξ, the same for all the natural vibration modes of the dam supported on rigid foundation rock with no impounded water, corresponds to a constant hysteretic damping factor of $\eta = 2\xi$ {7}.

In the initial dynamic analyses of the dam, a constant hysteretic damping factor of $\eta = 0.10$, which corresponds to a 5 percent viscous damping ratio in all natural vibration modes of the dam on rigid foundation rock with empty reservoir, was selected. Using this value of $\eta = 0.10$, very high tensile stresses were computed in the dam for QUAKE 1 and QUAKE 2 excitations, which are considered extremely strong earthquakes.

In view of the high stress levels due to strong earthquake ground motions, $\eta = 0.14$, which corresponds to a viscous damping ratio of 7 percent in all natural vibration modes of the dam (without impounded water) on rigid foundation rock, can reasonably be justified.

9.2 Wave Reflection Coefficient.

The boundary (bottom and sides) of a reservoir typically consists of layers of alluvium, silt, and other sedimentary materials. Hydrodynamic pressure waves impinging on such materials will partially reflect back into the water and be partially absorbed into the underlying layers of reservoir-boundary materials.

The absorptiveness of the reservoir-boundary materials is characterized by the wave reflection coefficient α, which is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a

normally propagating pressure wave incident on the reservoir boundary. A rigid reservoir boundary (α = 1.0) means that pressure waves are reflected from the reservoir boundary without attenuation; whereas a completely absorptive reservoir boundary (α = 0) means that normally propagating pressure waves are fully absorbed into the reservoir-boundary materials without reflection. The wave reflection coefficient α can be determined according to the following equation {2}:

$$\alpha = \frac{1 - qC}{1 + qC}$$

in which C is the velocity of pressure waves in water, q is the damping coefficient of the reservoir-boundary materials and is given by

$$q = \frac{\rho}{\rho_r \sqrt{E_r/\rho_r}}$$

where ρ is the mass density of water, E_r is the Young's modulus and ρ_r is the mass density of the materials at the reservoir boundary.

There are no experimental data available for the wave reflection coefficient of the materials present at the bottom and sides of the reservoir impounded by Englebright Dam. In the absence of such data, α is varied over a range to facilitate a study of the effects of wave absorption at the reservoir boundary. The α values considered in this investigation are α = 1.00, 0.90, 0.75 and 0.50. It was found that assumption of a completely rigid, non-absorptive reservoir boundary (α = 1.0) can lead to unrealistically large earthquake response for dams with impounded water {3}. An absorptive reservoir boundary that models the alluvium and sediments should give a more realistic estimate of the earthquake response of concrete arch dams. Professor Anil K. Chopra has recommended α = 0.90 or 0.75 with the larger value suggested for conservatism.

9.3 Hydrodynamic Effects.

Hydrodynamic pressures in the reservoir in excess of the usual hydrostatic pressures are developed due to earthquake ground motions and the deformations of the upstream face of the dam. The structural deformations are in turn affected by the hydrodynamic pressures acting on the dam. The hydrodynamic effects should be properly represented to recognize the dynamic interaction between the dam and the water.

Westergaard's classic work {8} on analysis of hydrodynamic pressures due to ground motion has some significant limitations. His solution is based on a straight rigid dam with vertical upstream face. Application of Westergaard's approach (in the form of added-mass formulation) to dynamic analysis of arch dam is therefore questionable due to the curvatures of the upstream face and flexibility of the dam.

One distinguishing feature of EACD's analysis procedure is the efficient formulations and computational procedures developed for evaluation of dam-water interaction during an earthquake. In this analysis procedure, the response of the dam with impounded water is affected by the hydrodynamic terms in the equations of motion for the dam. The hydrodynamic terms, which are determined from analysis of the reservoir finite element model, can be interpreted as modifying the properties of the dam by introducing an added mass, an added or subtracted hydrodynamic force, and an added damping {3}. Considering water compressibility, these hydrodynamic terms are functions of the excitation frequency. Note that analyses which neglect water compressibility generally lead to considerable error.

9.4 Fourier Transform Parameters.

The earthquake response of the dam is obtained by Fourier synthesis of the complex-valued frequency response functions using the Fast Fourier Transform (FFT) algorithm. To ensure that EACD computes accurate dynamic response, the Fourier Transform parameters must be properly selected. The parameters used in the FFT computations are as follows:

T = period of computation = 30.72 sec.

N = number of discrete time instants in computation period = $T/\Delta t$, in which Δt = 0.02 sec. is the time increment at which earthquake ground motions are defined. Hence, N = 30.72/0.02 = 1,536.

FMAX = maximum frequency represented = N/2T = 25.0 Hz.

The criteria that govern the selection of N (thus T) are discussed in reference {9}.

9.5 Cases of Dynamic Analysis.

Since there are no field-test data for the wave absorption coefficient α (recall that α depends upon the properties of the reservoir-boundary materials), the parameter is varied over a range in the present study to elucidate the effects of reservoir boundary absorption on the response of the dam. As mentioned in Section 9.2, the α values considered, in order of increasing absorptiveness, are α = 1.0 (rigid reservoir boundary), 0.90, 0.75, and 0.50. The earthquake response of the dam was analyzed for 16 cases, with different combinations of α and η, for the two earthquakes: QUAKE 1 and QUAKE 2. These cases are defined as shown in Table 9-1.

9.6 Response Results.

Because of the immense volume of response results generated from time-history earthquake analysis for the above-defined 16 cases, only a small portion of the results are presented in this report to highlight the important information that could aid the seismic evaluation of the dam. Unless stated otherwise, the stresses presented in this chapter are total stresses (initial static stresses due to dead weight of the dam and hydrostatic pressure are included), and are calculated for the following conditions: (1) the dam is subjected to simultaneous excitation of upstream, vertical, and cross-stream components of ground motion; and (2) the dam is supported on flexible foundation rock with full reservoir (gross pool elevation).

The maximum tensile stresses (arch, cantilever, and principal stresses) and the corresponding stress locations and times of occurrence are summarized in Table 9-2 for the dam subjected to QUAKE 1 (Cases 1 to 8), and in Table 9-3 for the dam subjected to QUAKE 2 (Cases 9 to 16). Maximum compressive stresses due to QUAKE 1 and QUAKE 2 are summarized in a similar manner in Tables 9-4 and 9-5. Five significant tensile stresses (in descending order) and the corresponding locations and times of occurrence for each of the cases are presented in Tables E-1 through E-4 in Appendix E. Plots of maximum tensile stresses (arch and cantilever stresses) versus the wave reflection coefficient for $\eta = 0.10$ and $\eta = 0.14$ are presented in Figures 9-5 through 9-8. Note from these figures that the response of the dam generally decreases with increasing wave absorption (α decreases from 1.0 to 0.5). This observation, which is consistent with intuition, results from the fact that reservoir boundary absorption reduces the added hydrodynamic force and introduces an added damping. As the reservoir boundary becomes more absorptive, i.e., as α decreases, the added damping at the fundamental resonant period generally increases because of increasing refraction of hydrodynamic pressure waves into the reservoir-boundary materials, and propagation of the pressure waves in the upstream direction {3}. However, it can be seen from Figures 9-7 and 9-8 that stress response in some cases increases as α decreases from 1.0 to 0.9, contrary to intuition. This "abnormal" behavior will be discussed in the following paragraphs, wherein the displacement and stress responses due to the three ground motion components of QUAKE 2 acting separately and simultaneously are examined.

The peak radial displacement at a selected dam crest nodal point and the maximum tensile values of arch and cantilever stresses (computed at Gauss Quadrature points), due to the three components of QUAKE 2 acting separately and simultaneously, are compiled in Table 9-6. Figures 9-9, 9-10, and 9-11 show the time history of radial displacement at nodal point 300 for $\alpha = 1.0$, 0.90, and 0.75, respectively, with $\eta = 0.10$. Note that the displacement and stress responses mentioned here are purely dynamic responses, with the static effects excluded. It is apparent from Table 9-6 (for QUAKE 2) that with increasing absorptiveness of the reservoir-boundary materials, the response of

the dam to upstream or vertical ground motions is reduced, as expected. For example, as α decreases from 1.0 to 0.9, the peak radial displacement due to the upstream ground motion decreases from 1.29 inch to 1.26 inch; the maximum arch stress decreases from 1,774 psi to 1,692 psi; and the maximum cantilever stress decreases from 823 psi to 774 psi (see Table 9-6a).

However, in some cases the response of the dam to cross-stream ground motion increases, contrary to intuition, with increasing wave absorption. As α decreases from 1.0 to 0.9, the peak radial displacement increases from 0.59 inch to 0.61 inch; the maximum arch stress increases from 289 psi to 308 psi; and the maximum cantilever stress increases slightly from 287 psi to 288 psi (see Table 9-6c). This abnormality, mentioned earlier in this section in terms of the total stress response due to the three components of QUAKE 2 acting simultaneously, was recognized in Dr. Fok's research investigation. The unexpected increase in response is attributed mainly to the increase in the fundamental resonant response to cross-stream ground motion because of reservoir boundary absorption {3}.

Comparing the displacement histories of Figures 9-9 through 9-11, it is of interest to note that the main effect of reservoir boundary absorption is to reduce the displacement peaks without significantly altering the frequency content of the response. Moreover, these figures serve to illustrate the significance of the three ground motion components in the total dynamic response of the dam, and to provide an indication of the phase differences between the responses to the individual components. It is evident that the contribution of the response to vertical or cross-stream ground motion is generally smaller than that due to upstream ground motion. Furthermore, the phase differences between the responses to the three components of ground motion are large enough to have a cancelling effect; i.e., the peak responses are not directly additive. The implication is that the peak response due to the three components acting simultaneously can be smaller than that due to upstream ground motion alone.

Tables 9-4 and 9-5 show that the maximum compressive stresses, even in the case where the reservoir boundary is conservatively assumed as rigid (α = 1.0), are well within the compressive strength of the dam concrete. Also, as mentioned in the preceding sections, use of η = 0.14 and a range of α values from 0.75 to 0.90 can rationally be justified in assessing the seismic performance of Englebright Dam. Therefore, attention will now be focused only on tensile stresses calculated with (1) η = 0.14, α = 0.90; and (2) η = 0.14, α = 0.75 for the dam subjected to QUAKE 1 and QUAKE 2 (see Cases 6, 7, 14, 15 in Table 9-1). The peak tensile values* of arch, cantilever, and principal stresses on the upstream and downstream faces along sections 6, 7, and 8 (central portion of the dam) are shown in Figures E-22 through E-27 in Appendix E. Similarly, the peak values of the same stress quantities on the mid-surface of the dam along sections 6, 7, and 8 are displayed in Figures E-28 through E-33. Peak (or envelope)† value here refers to the maximum value over time. Plots of the envelope tensile values of the major principal stresses and the corresponding minor principal stresses on the upstream and downstream faces are presented in Figures 9-12 through 9-19.

Stress contour plots are used to display the distributions of arch, cantilever, and principal stresses on the upstream and downstream faces of the dam. Contour plots of the envelope values of those stresses are shown in Figures 9-20 through 9-31. In addition, plots of the instantaneous values of the stresses at critical time instants are presented in Figures 9-32 through 9-51. The instantaneous stress contour plot can be thought of as a "stress picture" (snapshot) which changes with time in the course of earthquake shaking.

* In some cases where the total stress time-history lies entirely in the compression domain (see Figure E-21 in Appendix E), the more positive value of the two peak compressive stresses (negative stresses) associated with that time history is plotted. Tensile stress is understood to be positive, and compressive stress to be negative throughout this report.

† The words "peak" and "envelope" are used interchangeably in this report.

9.7 Areas of High Tensile Stresses.

With the aid of Figures E-22 through E-33 (Appendix E) and Figures 9-12 through 9-31 mentioned in the preceding section, regions of the dam where critical tensile stresses occur are identified. This identification will serve as a basis for evaluating the seismic safety of the dam subjected to the intense earthquake ground motions, namely, QUAKE 1 and QUAKE 2. Summarized in Table 9-7 are the maximum tensile stresses on the mid-face, upstream and downstream faces of the dam. Presented in Figures 9-52 through 9-93 are time histories of the major principal stress at the most-stressed locations. Time histories of arch and cantilever stresses at selected locations are displayed in Figures E-1 through E-20 in Appendix E. In general, QUAKE 2 causes larger peak stresses in the dam than does QUAKE 1; nevertheless, the stress distribution pattern developed in the dam due to QUAKE 1 is similar to that due to QUAKE 2, as is apparent from comparison of the principal stress plots. The areas of significant tensile stresses are discussed in the following paragraphs.

a. Upstream Face.

The highly stressed regions on the upstream face are the lower and upper parts in the vicinity of the crown cantilever. For example, the peak tensile stress due to QUAKE 1 ranges* from 1,128 psi to 1,295 psi at node 472; from 1,060 psi to 1,149 psi at node 481; from 1,087 psi to 1,104 psi at node 303; and from 1,032 psi to 1,035 psi at node 304 (see Figures 9-52 through 9-55). The peak tensile stress due to QUAKE 2 ranges from 1,564 psi to 1,682 psi at node 472; from 1,503 psi to 1,650 psi at node 481; from 1,638 psi to 1,791 psi at node 303; and from 1,629 psi to 1,774 psi at node 304 (see Figures 9-73 through 9-76). Figures 9-94 through 9-97 show the time histories of the radial, vertical, and tangential dynamic displacements at dam crest nodal point 303.

* The range of computed stresses corresponds to two values of the wave absorption coefficient: $\alpha = 0.75$ and 0.90.

b. <u>Downstream Face</u>.

The most-stressed areas on the downstream face are the upper central portion, and the regions in the neighborhood of sections 6 and 9 away from the crown cantilever. For instance, the peak tensile stress due to QUAKE 1 ranges from 722 psi to 762 psi at node 22; from 786 psi to 846 psi at node 149; and from 686 psi to 694 psi at node 114 (see Figures 9-56 through 9-58). The peak tensile stress due to QUAKE 2 ranges from 951 psi to 1,050 psi at node 22; from 1,278 psi to 1,397 psi at node 149; and from 1,053 psi to 1,151 psi at node 114 (see Figures 9-77 through 9-79).

c. <u>Mid-Surface</u>.

Significant tensile stresses are confined to the upper central areas of the mid-surface inside the dam near the crown cantilever (see Figures E-28 through E-33 in Appendix E). For example, the peak tensile stress due to QUAKE 1 ranges from 867 psi to 869 psi at node 217; from 784 psi to 821 psi at node 218; from 637 psi to 667 psi at node 227; and from 591 psi to 623 psi at node 228 (see Figures 9-59 through 9-62). The peak tensile stress due to QUAKE 2 ranges from 1,283 psi to 1,404 psi at node 217; from 1,154 psi to 1,256 psi at node 218; from 917 psi to 1,007 psi at node 227; and from 876 psi to 958 psi at node 228 (see Figures 9-80 through 9-83).

d. <u>Gauss Quadrature Points</u>.

The analysis indicates relatively large tensile stresses in the upper central part of the dam, at Gauss Quadrature points inside such dam-crest elements as elements 8, 9, and 10 (see Figure 9-1, Tables E-1 and E-3 in Appendix E). For example, the peak tensile stress due to QUAKE 1 ranges from 917 psi to 919 psi at stress location 4 in element 9; from 872 psi to 907 psi at stress location 1 in element 9; from 868 psi to 880 psi at stress location 1 in element 8; from 746 psi to 803 psi at stress location 4 in element 8; and from 727 psi to 774 psi at stress location 4 in element 10 (see Figures 9-63 through 9-67). The peak tensile stress due to QUAKE 2

ranges from 1,415 psi to 1,546 psi at stress location 4 in element 9; from 1,351 psi to 1,472 psi at stress location 1 in element 9; from 1,265 psi to 1,388 psi at stress location 1 in element 8; from 1,047 psi to 1,154 psi at stress location 4 in element 8; and from 1,040 psi to 1,129 psi at stress location 4 in element 10 (see Figures 9-84 through 9-88).

It is worth noting that the stresses in the lower parts are less significant as compared to those in the upper central part. For instance, the peak tensile stress due to QUAKE 1 ranges from 540 psi to 593 psi at stress location 2 in element 56; from 512 psi to 593 psi at stress location 3 in element 56; from 482 psi to 526 psi at stress location 2 in element 52; from 459 psi to 548 psi at stress location 3 in element 51; and from 489 psi to 583 psi at stress location 2 in element 55 (see Figures 9-68 through 9-72). The peak tensile stress due to QUAKE 2 ranges from 782 psi to 867 psi at stress location 2 in element 56; from 733 psi to 813 psi at stress location 3 in element 56; from 708 psi to 792 psi at stress location 2 in element 52; from 701 psi to 764 psi at stress location 3 in element 51; and from 681 psi to 742 psi at stress location 2 in element 55 (see Figures 9-89 through 9-93).

9.8 Maximum Repeatable Stress Level.

It is becoming more generally recognized that damage to a concrete dam is not predominantly controlled by the transitory peak stress response. Tensile stresses which are repeated several times during an earthquake can be more damaging than a single large peak stress. In this study a maximum repeatable stress level is defined as the maximum stress value which is exceeded by six short duration excursions during the ground shaking (the number of repetitions being an arbitrary number that has been used in other investigations). Attempts have been made to identify this stress level for arch and cantilever tensile stresses at highly stressed locations on the faces of the dam. The results are compiled in Table 9-8.

CHAPTER 10

EVALUATION OF DAM SAFETY

10.1 Apparent Tensile Strength.

In this study, the stress response is based on a linearly elastic analysis performed on the dam concrete that actually behaves nonlinearly. In judging the safety of the dam, it is appropriate to compare the computed tensile stresses with the "apparent tensile strength" of the concrete {10, 11}. The apparent tensile strength is equal to the splitting tensile strength augmented by a factor of 1.3 to take into account the nonlinearity of concrete. Applying the 1.3 factor to the actual tensile strength values of 466 psi and 624 psi (see Section 4.1), the apparent static tensile strength is approximately equal to 600 psi and the apparent seismic tensile strength is equal to 800 psi.

10.2 Static Performance.

Based on the static response analysis of Englebright Dam for the usual loading condition (full pool condition), the maximum tensile and compressive stresses are well below the strength capacity of the dam concrete. As was noted in Section 8.2, the largest tensile stress is 207 psi on the upstream face, which is about 35 percent of the apparent static tensile strength (600 psi) of the concrete. The largest compressive stress is 658 psi on the downstream face, which is 10 percent of the static compressive strength (6,530 psi). It is therefore concluded that the dam is structurally capable of withstanding the usual loading condition.

For the case of unusual loading (surcharge loading corresponding to PMF pool elevation 557.7 feet), the largest tensile stress is 586 psi on the upstream face, which falls just within the apparent static tensile strength (600 psi) of the concrete. This high tensile stress is very localized according to the GTSTRUDL analysis. The largest compressive stress is 752 psi on the downstream face, which is about 12 percent of the static compressive

strength (6,530 psi). It is therefore concluded that the dam is structurally capable of withstanding the unusual loading condition.

10.3 Criteria for Seismic Performance.

The actual performance of concrete arch dams during past earthquakes is known to be generally satisfactory. For example, the Pacoima Dam which was subject to very intense ground motions during the 1971 San Fernando earthquake only suffered the opening of a joint between the arch and the left thrust block, with no other apparent damage to the arch structure itself. This historical event provides some evidence in support of the generally accepted notion that concrete arch dams have good endurance in relation to strong seismic action.

Currently, no standard criteria are available for evaluating the seismic safety of arch dams on the basis of dynamic finite element analysis. Traditional criteria setting forth factors of safety (based on concrete strengths) should not be used because they do not provide a basis for assessing the overall safety of an arch dam. The damage that concrete dams may experience due to intense earthquake ground shaking is primarily in the form of cracking in areas subjected to large tensile stresses. Ideally, the prevalent level of computed tensile stress from linearly elastic analyses should be no greater than the apparent tensile strength of the dam concrete. However, it is usually acceptable to allow tensile stresses exceeding the apparent tensile strength of the concrete at a limited number of locations, for brief instants of time, for earthquakes having remote probabilities of occurrence. Computed compressive stresses, on the other hand, should not be allowed to exceed the compressive strength. A dam may undergo some cracking as a result of large tensile stresses induced by an earthquake. However, this does not imply that the dam is unsafe. If cracking should occur, structural failure or instability would not necessarily develop either during or after the seismic event. In evaluating the seismic safety of the dam, the most crucial factors that need to be considered are: (1) the potential for tensile cracking through the entire thickness of the dam during the earthquake, and

(2) the dam's capability of resisting the static loads (self weight of the dam plus water pressure) which remain after the earthquake has ended.

10.4 Evaluation of Seismic Safety.

As mentioned earlier in Section 9.6, the maximum compressive stresses caused by QUAKE 1 and QUAKE 2 (see Tables 9-4 and 9-5) are well within the measured dynamic compressive strength of the dam concrete (6,664 psi), thus precluding compressive failure. Attention will now be directed to the large tensile arch stresses and cantilever stresses indicated in the earthquake response analysis. Based on the apparent seismic tensile strength of 800 psi and the stress results presented in Chapter 9, the extent and distribution of likely cracking can be predicted.

The envelope values of tensile arch stresses computed for QUAKE 1 and QUAKE 2 excitations are displayed in the form of contour plots in Figures 9-20, 9-21, 9-26, and 9-27. With $\eta = 0.14$ and $\alpha = 0.90$, the largest tensile arch stress produced by QUAKE 1 is 1,104 psi and that produced by QUAKE 2 is 1,790 psi, significantly in excess of the apparent seismic tensile strength (see Tables 9-2 and 9-3). The maximum repeatable arch stress level that is exceeded six times during an earthquake is 891 psi for QUAKE 1, and 851 psi for QUAKE 2 (see Table 9-8). These relatively high tensile stresses could not be realized in the actual dam because the horizontal tensile arch stresses would be effectively dissipated by the opening of the vertical contraction joints. An opened joint would quickly be closed again because the stresses in the dam subjected to earthquake vibration are oscillatory in nature and reverse direction many times. Joint opening, followed by immediate closing during the ground shaking, would have no adverse effect on the structural integrity of the dam but would reduce the tendency of concrete cracking. Furthermore, this cycling mechanism (opening and closing) of the joints would promote dissipation of the stored dynamic energy and hence reduce the dynamic response of the dam. In view of the above-mentioned effects associated with the vertical contraction joints, the large tensile stresses acting in the arch direction are not considered to be of any serious consequence.

The envelope values of tensile cantilever stresses computed for QUAKE 1 and QUAKE 2 excitations are presented as contour plots in Figures 9-22, 9-23, 9-28, and 9-29. Based on the analysis with $\eta = 0.14$ and $\alpha = 0.90$, the largest tensile cantilever stress induced by QUAKE 1 is 1,222 psi on the upstream face, and that induced by QUAKE 2 is 1,650 psi on the upstream face (see Tables 9-2 and 9-3). The maximum repeatable cantilever stress level on this face is 949 psi for QUAKE 1, and 813 psi for QUAKE 2 (see Table 9-8). It can be seen by comparison of Figures 9-22 with 9-23 and 9-28 with 9-29 that the cantilever stresses developed on the upstream face are much larger than those on the downstream face. The computed surface stresses acting in the cantilever direction are so severe that cracking of concrete would undoubtedly occur under earthquake conditions. Cracking is confined to areas over the upstream and downstream faces where sufficiently high tensile cantilever stresses are developed once or possibly a few times during the intense ground motion. Based on comparison of the envelope values of tensile cantilever stresses with the 800 psi apparent tensile strength capacity, the areas where cracking can possibly occur are identified and indicated in Figures 9-22, 9-28, and 9-29.

Instantaneous cantilever stress distributions at critical time instants shown in Figures 9-44 through 9-51 provide a more complete understanding of the cantilever-stress situation developed in the dam. It is obvious from these figures that tensile stresses at locations on one face of the dam is accompanied by compressive stresses at the corresponding locations on the other face at a certain time instant. For example, the earthquake response analysis indicates that at time $t = 3.12$ seconds, a tensile stress of 1,424 psi due to QUAKE 2 develops at the upstream base (node 486) of the crown cantilever while a compressive stress of 1,409 psi is indicated at the downstream base (node 205). At a later time $t = 4.66$ seconds, a compressive stress of 1,377 psi develops at the upstream base, and a tensile stress of 605 psi at the downstream base. This example serves to demonstrate that the section undergoes flexural behavior, and most importantly, that tensile stresses do not develop concurrently through the entire thickness. Thus cracking would essentially be a surface phenomenon with possibly small depth

of penentration inside the dam, but would not extend through the entire thickness. The compressive static stresses due to dead weight of the dam and hydrostatic pressure, which act during and after the ground shaking, will provide significant confining forces to close some earthquake-induced cracks after the shaking stops. These cracks are not regarded as potentially damaging and will not impair the ability of the dam to contain the impounded water.

The large inertia forces and stresses developed in the dam might suggest some cause for concern for structural stability. These large forces and stresses only act for very short durations of time and reverse in direction many times during the earthquake, as would be expected from the oscillatory nature of the dam response. On this basis, it is expected that structural instability would not occur, and that the structure would be structurally capable of resisting the static loads after the earthquake.

It should be emphasized that the analysis in this study is based on a linearly elastic model wherein cracking and subsequent non-linear behavior are not considered. The dynamic response of a dam is known to be influenced significantly by cracking. Recognizing the reduction in stiffness of the dam due to cracking and the increased energy dissipation in cracking, it is evident that the nonlinearities associated with the cracks will reduce the extent and severity of the highly stressed regions below that revealed by the linearly elastic analysis. In other words, the cracking induced by QUAKE 1 or QUAKE 2 will be less extensive than the probable cracking zones shown in Figures 9-22, 9-28, and 9-29, which are predicted from the results of linear analysis.

Further conservatism in the computed stresses is introduced by the assumption that the cross-stream component of earthquake ground motion has the same intensity of the upstream ground motion accelerogram (see Appendix C).

In conclusion, the dam is capable of surviving QUAKE 1 and QUAKE 2 in such a way that no failure triggering a sudden, catastrophic release of water

will occur, and that life and property downstream will not be endangered. If the postulated M.C.E. events (QUAKE 1 and QUAKE 2) were to occur during a high pool condition, some surface cracking would occur, but the extent of cracking will be less widespread than that predicted by the linearly elastic analysis. This is acceptable performance for the Maximum Credible Earthquake considered in this investigation, which, as stressed by Dr. Bolt, "... is very unlikely within the lifetime of the dam and its adoption would provide an upper bound to feasible ground motions for engineering analysis."

10.5 Concluding Remarks.

Seismic evaluations of concrete dams to date are largely based on linearly elastic analysis. In view of the fact that extreme earthquakes can create stresses outside the elastic range of deformation, it seems desirable to develop nonlinear analysis procedure which takes into consideration the nonlinear material properties of concrete. This points to the need for a comprehensive experimental research program to better determine the constitutive and strength properties of multi-axially loaded mass concrete under dynamic, reversible, cyclic strains and stresses representative of earthquake vibration conditions. It is only after these mechanical properties of concrete are better determined and incorporated into a reliable nonlinear analysis procedure that the extent of cracking and its implications to the safety of a dam can be determined with high degree of confidence.

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TABLE 1-1

ENGLEBRIGHT DAM

General

Stream	Yuba River
Drainage area above dam	1,100 square miles

Dam

Type	Concrete arch, overflow
Maximum height	260 feet
Elevation, top of dam	542 feet
Elevation, outlet tunnel	438 feet
Total crest length	1,142 feet
Diameter of outlet tunnel	9 feet

Reservoir

Gross Pool:	
Elevation (USGS datum)	526.99 feet
Water storage capacity	70,000 acre-feet
Debris storage capacity	118,000 cubic yards
Surface area	815 acres
Shoreline	24 miles
Length	9 miles
Minimum Operational Pool:	
Elevation	450 feet
Capacity	25,000 acre-feet
Surface area	387 acres
Length	5 miles

TABLE 1-2

YUBA RIVER AT ENGLEBRIGHT DAM

DATE OF MAJOR FLOOD	PEAK FLOW, cfs
Nov 1950	109,000
Dec 1955*	148,000
Oct 1962	91,700
Feb 1963*	150,000
Dec 1964*	171,000
Jan 1970	93,500
Jan 1980	45,000
Feb 1986	99,200

*Capacity of the central overflow section of spillway was exceeded, based on spillway capacity curves.

TABLE 1-3

PROBABLE MAXIMUM FLOOD

FLOW IN cfs	NUMBER OF HOURS EXCEEDING FLOW
200,000	41
300,000	25
400,000	4

TABLE 6-1: COMPUTED MODAL FREQUENCIES OF DAM ON RIGID FOUNDATION ROCK WITH EMPTY RESERVOIR

MODE NO.	NATURAL FREQUENCIES (CYCLES/SEC.)							DIFFERENCE IN MODAL FREQUENCY BETWEEN 1L20ND MODEL AND 2L20ND MODEL (GTSTRUDL-CONSISTENT MASS)	DIFFERENCE IN MODAL FREQUENCY BETWEEN 1L20ND MODEL AND 8NTSD MODEL (EACD-CONSISTENT MASS)	DIFFERENCE IN MODAL FREQUENCY BETWEEN 1L20ND MODEL AND 1L8ND MODEL (GTSTRUDL-CONSISTENT MASS)
	1L8ND MODEL		1L20ND MODEL			2L20ND MODEL	8NTSD MODEL			
	GTSTRUDL		GTSTRUDL		EACD	GTSTRUDL	EACD			
	LUMPED MASS	CONSISTENT MASS	LUMPED MASS	CONSISTENT MASS	CONSISTENT MASS	CONSISTENT MASS	CONSISTENT MASS	PERCENT (%)	PERCENT (%)	PERCENT (%)
1	5.813	6.118	5.453	5.662	5.664	5.620	5.508	0.75	2.75	8.05
2	6.381	6.905	6.175	6.479	6.479	6.411	5.712	1.06	11.84	6.58
3	8.311	9.361	7.752	8.225	8.235	8.108	7.059	1.44	14.28	13.81
4	10.590	12.345	8.090	8.658	8.656	8.534	8.313	1.45	3.96	42.58
5	12.391	13.543	10.213	11.073	11.072	10.912	9.885	1.48	10.72	22.31
6	12.870	14.065	11.200	12.111	12.112	11.946	11.750	1.38	2.99	16.13
7	13.021	15.766	12.059	13.402	13.396	13.144	11.940	1.96	10.87	17.64
8	14.702	16.353	12.607	13.618	13.608	13.267	12.619	2.65	7.27	20.08
9	15.252	17.631	12.789	14.158	14.157	14.029	13.818	0.92	2.39	24.53
10	15.580	18.839	14.154	15.551	15.553	15.405	15.018	0.95	3.44	21.14

DEFINITIONS:

1L8ND MODEL: 1 - LAYER - 8 - NODE SOLID FINITE ELEMENT DAM MODEL
1L20ND MODEL: 1 - LAYER - 20 - NODE SOLID FINITE ELEMENT DAM MODEL
2L20ND MODEL: 2 - LAYER - 20 - NODE SOLID FINITE ELEMENT DAM MODEL
8NTSD MODEL: 8 - NODE THICK SHELL FINITE ELEMENT DAM MODEL

TABLE 6-2: FEATURES OF THE FOUR FINITE ELEMENT DAM MODELS

FINITE ELEMENT DAM MODEL	FINITE ELEMENTS COMPOSING THE DAM MODEL	TOTAL NUMBER OF DAM NODES	TOTAL NUMBER OF FIXED DAM NODES FOR THE CASE OF COMPLETE CONSTRAINTS AT DAM-FOUNDATION INTERFACE	TOTAL NUMBER OF DEGREES OF FREEDOM FOR THE CASE OF COMPLETE CONSTRAINTS AT DAM-FOUNDATION INTERFACE
1L8ND MODEL	1 LAYER OF FINITE ELEMENTS THROUGH THE THICKNESS OF THE DAM 63 EIGHT-NODE SOLID ELEMENTS AND 2 SIX-NODE PRISMATIC SOLID ELEMENTS	174	58	348
1L20ND MODEL	1 LAYER OF FINITE ELEMENTS THROUGH THE THICKNESS OF THE DAM 55 TWENTY-NODE SOLID ELEMENTS AND 2 SIX-NODE PRISMATIC SOLID ELEMENTS	487	133	1062
2L20ND MODEL	2 LAYERS OF FINITE ELEMENTS THROUGH THE THICKNESS OF THE DAM 110 TWENTY-NODE SOLID ELEMENTS AND 4 SIX-NODE PRISMATIC SOLID ELEMENTS	768	213	1665
8NTSD MODEL	1 LAYER OF FINITE ELEMENTS THROUGH THE THICKNESS OF THE DAM 55 EIGHT-NODE SHELL ELEMENTS AND 2 THREE-NODE PRISMATIC SHELL ELEMENTS	206	53	765

TABLE 6-3a: FEATURES OF DAM MESH AND FOUNDATION MESH IN EACD ANALYSIS

FOUNDATION ROCK	NUMBER OF DAM ELEMENTS	NUMBER OF FREE DAM NODES	NUMBER OF FIXED DAM NODES	TOTAL NUMBER OF DAM NODES	NUMBER OF DEGREES OF FREEDOM OF DAM MESH	NUMBER OF FOUNDATION ELEMENTS	NUMBER OF FREE FOUNDATION NODES	NUMBER OF FIXED FOUNDATION NODES	TOTAL NUMBER OF FOUNDATION NODES	NUMBER OF DEGREES OF FREEDOM OF FOUNDATION MESH
RIGID	57	354	133	487	1062	N/A	N/A	N/A	N/A	N/A
FLEXIBLE	57	487	0	487	1461	468	619	162	781	1857

TABLE 6-3b: FEATURES OF THE FINITE ELEMENT MODEL IN GTSTRUDL ANALYSIS

FOUNDATION ROCK	NUMBER OF DAM ELEMENTS	NUMBER OF FOUNDATION ELEMENTS	NUMBER OF FREE NODES OF THE FINITE ELEMENT MODEL	NUMBER OF FIXED NODES OF THE FINITE ELEMENT MODEL	TOTAL NUMBER OF NODES OF THE FINITE ELEMENT MODEL	NUMBER OF DEGREES OF FREEDOM OF THE FINITE ELEMENT MODEL
RIGID	57	N/A	354	133	487	1062
FLEXIBLE	57	468	973	162	1135	2919

TABLE 7-1 COMPUTED NATURAL FREQUENCIES OF THE FIRST 15 MODES OF VIBRATION OF THE DAM WITH EMPTY RESERVOIR.

MODE NUMBER	COMPUTED NATURAL FREQUENCIES OF THE DAM ON RIGID FOUNDATION ROCK WITH EMPTY RESERVOIR (CYCLES/SEC.)	COMPUTED NATURAL FREQUENCIES OF THE DAM-FOUNDATION SYSTEM WITH EMPTY RESERVOIR NOTE: FOUNDATION ROCK IS FLEXIBLE (CYCLES/SEC.)	DECREASE IN NATURAL FREQUENCY DUE TO THE EFFECTS OF FOUNDATION FLEXIBILITY PERCENT (%)
1	5.664	5.319	6.09
2	6.479	5.430	16.19
3	8.235	6.777	17.70
4	8.656	8.058	6.91
5	11.072	9.670	12.66
6	12.112	11.588	4.33
7	13.396	11.628	13.20
8	13.608	11.937	12.28
9	14.157	13.708	3.17
10	15.553	14.291	8.11
11	16.297	14.332	12.06
12	16.807	15.468	7.97
13	17.571	16.075	8.51
14	18.573	16.727	9.94
15	19.608	18.128	7.55

TABLE 8-1: SUMMARY OF STATIC TENSILE STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESERVOIR IS FULL WITH POOL ELEVATION = 527 FEET. STATIC STRESSES ARE COMPUTED AT THE DAM NODAL POINTS USING EACD.

STATIC ARCH STRESS		STATIC CANTILEVER STRESS		STATIC PRINCIPAL STRESS	
TENSILE STRESS (PSI)	DAM NODAL POINT	TENSILE STRESS (PSI)	DAM NODAL POINT	TENSILE STRESS (PSI)	DAM NODAL POINT
STATIC RESPONSE OF THE DAM ON FLEXIBLE FOUNDATION ROCK					
57	472	191	472	207	472
48	482	172	486	173	486
46	487	146	487	148	487
44	378	122	469	129	482
35	469	115	463	124	469
STATIC RESPONSE OF THE DAM ON RIGID FOUNDATION ROCK					
80	472	244	472	262	472
62	482	195	486	196	486
48	487	147	487	149	487
61	378	164	469	166	482
53	469	163	463	167	469

TABLE 9-1: CASES OF DAM-WATER-FOUNDATION SYSTEM ANALYZED

CASE	CONSTANT HYSTERETIC DAMPING FACTOR η	WAVE REFLECTION COEFFICIENT α	EARTHQUAKE EXCITATION
1	0.10	1.00	QUAKE 1
2	0.10	0.90	QUAKE 1
3	0.10	0.75	QUAKE 1
4	0.10	0.50	QUAKE 1
5	0.14	1.00	QUAKE 1
6	0.14	0.90	QUAKE 1
7	0.14	0.75	QUAKE 1
8	0.14	0.50	QUAKE 1
9*	0.10	1.00	QUAKE 2
10*	0.10	0.90	QUAKE 2
11*	0.10	0.75	QUAKE 2
12	0.10	0.50	QUAKE 2
13	0.14	1.00	QUAKE 2
14	0.14	0.90	QUAKE 2
15	0.14	0.75	QUAKE 2
16	0.14	0.50	QUAKE 2

*CASES 9, 10, AND 11 ARE ANALYZED FOR FOUR EXCITATIONS: UPSTREAM GROUND MOTION ONLY, VERTICAL GROUND MOTION ONLY, CROSS-STREAM GROUND MOTION ONLY, AND ALL THREE GROUND MOTION COMPONENTS ACTING SIMULTANEOUSLY. THE OTHER CASES ARE ANALYZED ONLY FOR ALL THREE COMPONENTS ACTING SIMULTANEOUSLY.

TABLE 9-2: SUMMARY OF MAXIMUM TENSILE STRESSES (ARCH, CANTILEVER, AND PRINCIPAL) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

CASE	η	α	MAXIMUM TENSILE ARCH STRESS (*PSI*)	TIME OF OCCURRENCE (*SEC.*)	LOCATION WHERE MAXIMUM TENSILE ARCH STRESS OCCURS		MAXIMUM TENSILE CANTILEVER STRESS (*PSI*)	TIME OF OCCURRENCE (*SEC.*)	LOCATION WHERE MAXIMUM TENSILE CANTILEVER STRESS OCCURS		MAXIMUM TENSILE PRINCIPAL STRESS (*PSI*)	TIME OF OCCURRENCE (*SEC.*)	LOCATION WHERE MAXIMUM TENSILE PRINCIPAL STRESS OCCURS	
					ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER
QUAKE I			STRESSES COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS											
1	0.10	1.00	1295	7.70	9	4	785	7.58	56	2	1297	7.70	9	4
2	0.10	0.90	1091	7.24	9	4	712	7.58	56	2	1091	7.24	9	4
3	0.10	0.75	1052	7.24	9	4	639	7.58	56	2	1052	7.24	9	4
4	0.10	0.50	932	7.24	9	4	585	7.56	56	3	933	7.24	9	4
5	0.14	1.00	1165	6.78	9	4	687	4.92	56	3	1167	6.78	9	4
6	0.14	0.90	917	7.24	9	4	593	4.92	56	3	917	7.24	9	4
7	0.14	0.75	919	7.26	9	4	520	7.58	56	2	919	7.26	9	4
8	0.14	0.50	812	7.26	9	4	494	7.56	56	3	812	7.26	9	4
QUAKE I			STRESSES COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS											
1	0.10	1.00	1629	7.70	-	304	1497	7.58	-	484	1630	7.70	-	304
2	0.10	0.90	1319	5.04	-	303	1382	7.58	-	484	1440	7.58	-	484
3	0.10	0.75	1230	7.26	-	303	1257	7.58	-	484	1314	7.58	-	484
4	0.10	0.50	1106	7.26	-	303	1133	7.56	-	486	1172	7.58	-	484
5	0.14	1.00	1417	6.78	-	304	1391	4.92	-	472	1477	4.92	-	472
6	0.14	0.90	1104	5.04	-	303	1222	4.92	-	472	1295	4.92	-	472
7	0.14	0.75	1086	7.26	-	303	1065	4.92	-	472	1128	4.92	-	472
8	0.14	0.50	981	7.26	-	303	991	6.64	-	472	1044	6.64	-	472

*EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

TABLE 9-3: SUMMARY OF MAXIMUM TENSILE STRESSES (ARCH, CANTILEVER, AND PRINCIPAL) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

CASE	η	α	MAXIMUM TENSILE ARCH STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM TENSILE ARCH STRESS OCCURS		MAXIMUM TENSILE CANTILEVER STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM TENSILE CANTILEVER STRESS OCCURS		MAXIMUM TENSILE PRINCIPAL STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM TENSILE PRINCIPAL STRESS OCCURS	
					ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER
QUAKE 2			STRESSES COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS											
9	0.10	1.00	1758	4.66	9	4	1053	4.78	56	2	1758	4.66	9	4
10	0.10	0.90	1831	4.66	9	4	1035	4.78	56	2	1831	4.66	9	4
11	0.10	0.75	1664	4.66	9	4	926	4.78	56	2	1665	4.66	9	4
12	0.10	0.50	1401	4.66	9	4	762	4.78	56	2	1402	4.66	9	4
13	0.14	1.00	1434	4.66	9	4	825	4.78	56	2	1435	4.66	9	4
14	0.14	0.90	1546	4.66	9	4	851	4.78	56	2	1546	4.66	9	4
15	0.14	0.75	1415	4.66	9	4	767	4.78	56	2	1415	4.66	9	4
16	0.14	0.50	1202	4.66	9	4	637	4.78	56	2	1202	4.66	9	4
QUAKE 2			STRESSES COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS											
9	0.10	1.00	2032	4.66	-	304	2015	4.78	-	484	2062	4.78	-	484
10	0.10	0.90	2124	4.66	-	303	1973	4.78	-	484	2126	4.66	-	303
11	0.10	0.75	1929	4.66	-	303	1784	4.78	-	484	1931	4.66	-	303
12	0.10	0.50	1624	4.66	-	304	1499	3.12	-	472	1625	4.66	-	303
13	0.14	1.00	1658	4.66	-	304	1735	3.12	-	472	1820	3.12	-	472
14	0.14	0.90	1790	4.66	-	303	1650	4.78	-	484	1791	4.66	-	303
15	0.14	0.75	1636	4.66	-	303	1506	4.78	-	484	1638	4.66	-	303
16	0.14	0.50	1392	4.66	-	304	1375	3.12	-	472	1439	3.12	-	472

*EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

TABLE 9-4: SUMMARY OF MAXIMUM COMPRESSIVE STRESSES (ARCH, CANTILEVER, AND PRINCIPAL) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

CASE	η	α	MAXIMUM COMPRESSIVE ARCH STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE ARCH STRESS OCCURS		MAXIMUM COMPRESSIVE CANTILEVER STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE CANTILEVER STRESS OCCURS		MAXIMUM COMPRESSIVE PRINCIPAL STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE PRINCIPAL STRESS OCCURS	
					ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER
QUAKE I			STRESSES COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS											
1	0.10	1.00	1772	5.38	9	4	983	7.58	57	8	1774	5.38	9	4
2	0.10	0.90	1842	5.38	9	4	952	7.58	57	7	1842	5.38	9	4
3	0.10	0.75	1715	5.38	9	4	903	7.58	57	7	1716	5.38	9	4
4	0.10	0.50	1553	5.38	9	4	843	7.58	57	7	1553	5.38	9	4
5	0.14	1.00	1567	5.38	9	4	867	7.58	57	7	1568	5.38	9	4
6	0.14	0.90	1593	5.38	9	4	855	5.38	57	8	1594	5.38	9	4
7	0.14	0.75	1504	5.38	9	4	819	5.38	57	8	1505	5.38	9	4
8	0.14	0.50	1380	5.38	9	4	768	5.38	57	8	1380	5.38	9	4
QUAKE I			STRESSES COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS											
1	0.10	1.00	2093	5.14	-	303	1693	5.14	-	191	2094	5.14	-	303
2	0.10	0.90	2114	5.38	-	303	1608	5.38	-	191	2117	5.38	-	303
3	0.10	0.75	1977	5.38	-	303	1520	5.38	-	191	1979	5.38	-	303
4	0.10	0.50	1796	5.38	-	303	1400	5.38	-	191	1797	5.38	-	303
5	0.14	1.00	1849	6.88	-	303	1520	6.66	-	191	1849	6.88	-	303
6	0.14	0.90	1813	5.38	-	303	1435	5.14	-	191	1816	5.38	-	303
7	0.14	0.75	1719	5.38	-	303	1361	5.38	-	191	1722	5.38	-	303
8	0.14	0.50	1585	7.36	-	303	1270	5.38	-	191	1585	7.36	-	303

*EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

TABLE 9-5: SUMMARY OF MAXIMUM COMPRESSIVE STRESSES (ARCH, CANTILEVER, AND PRINCIPAL) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

CASE	η	α	MAXIMUM COMPRESSIVE ARCH STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE ARCH STRESS OCCURS		MAXIMUM COMPRESSIVE CANTILEVER STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE CANTILEVER STRESS OCCURS		MAXIMUM COMPRESSIVE PRINCIPAL STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	LOCATION WHERE MAXIMUM COMPRESSIVE PRINCIPAL STRESS OCCURS	
					ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER			ELEMENT NUMBER	STRESS LOCATION* OR NODAL POINT NUMBER
QUAKE 2			STRESSES COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS											
9	0.10	1.00	2183	3.12	8	1	1256	4.78	57	8	2184	3.12	8	1
10	0.10	0.90	2197	4.78	9	4	1230	4.78	57	8	2199	4.78	9	4
11	0.10	0.75	1987	4.78	9	4	1147	4.78	57	8	1988	4.78	9	4
12	0.10	0.50	1744	3.12	8	1	1012	4.78	57	8	1744	3.12	8	1
13	0.14	1.00	2063	3.12	8	1	1075	4.78	57	8	2063	3.12	8	1
14	0.14	0.90	1930	3.12	8	1	1081	4.78	57	8	1930	3.12	8	1
15	0.14	0.75	1779	3.12	8	1	1017	4.78	57	8	1779	3.12	8	1
16	0.14	0.50	1621	3.12	8	1	910	4.78	57	8	1621	3.12	8	1
QUAKE 2			STRESSES COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS											
9	0.10	1.00	2690	3.12	-	303	1983	4.78	-	195	2690	3.12	-	303
10	0.10	0.90	2563	4.78	-	304	1947	4.78	-	195	2564	4.78	-	304
11	0.10	0.75	2355	3.12	-	303	1799	4.78	-	195	2355	3.12	-	303
12	0.10	0.50	2144	3.12	-	303	1640	3.12	-	191	2144	3.12	-	303
13	0.14	1.00	2508	3.12	-	303	1842	3.12	-	191	2508	3.12	-	303
14	0.14	0.90	2365	3.12	-	303	1727	3.12	-	191	2365	3.12	-	303
15	0.14	0.75	2176	3.12	-	303	1635	3.12	-	191	2176	3.12	-	303
16	0.14	0.50	1969	3.12	-	303	1539	3.12	-	191	1969	3.12	-	303

*EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

TABLE 9-6: SUMMARY OF DYNAMIC RESPONSES* OF THE DAM, ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), TO QUAKE 2, η = 0.10

CASE	α	PEAK RADIAL DISPLACEMENT AT DAM CREST NODAL POINT 300 *(INCHES)*	MAXIMUM TENSILE ARCH STRESS *(PSI)*	LOCATION WHERE MAXIMUM TENSILE ARCH STRESS OCCURS		MAXIMUM TENSILE CANTILEVER STRESS *(PSI)*	LOCATION WHERE MAXIMUM TENSILE CANTILEVER STRESS OCCURS	
				ELEMENT NUMBER	STRESS LOCATION NUMBER		ELEMENT NUMBER	STRESS LOCATION NUMBER
(a) RESPONSE TO UPSTREAM COMPONENT OF QUAKE 2								
9	1.00	1.29	1774	9	4	823	56	3
10	0.90	1.26	1692	9	4	774	56	3
11	0.75	1.21	1576	9	4	701	56	3
(b) RESPONSE TO VERTICAL COMPONENT OF QUAKE 2								
9	1.00	0.55	751	9	4	353	56	3
10	0.90	0.47	748	9	4	346	56	3
11	0.75	0.36	589	9	4	296	56	3
(c) RESPONSE TO CROSS-STREAM COMPONENT OF QUAKE 2								
9	1.00	0.59	289	10	4	287	57	2
10	0.90	0.61	308	10	4	288	57	2
11	0.75	0.63	312	10	4	287	57	2
(d) RESPONSE TO UPSTREAM, VERTICAL, AND CROSS-STREAM COMPONENTS, SIMULTANEOUSLY, OF QUAKE 2								
9	1.00	1.32	2067	9	4	1060	56	2
10	0.90	1.21	2141	9	4	1042	56	2
11	0.75	1.15	1974	9	4	933	56	2

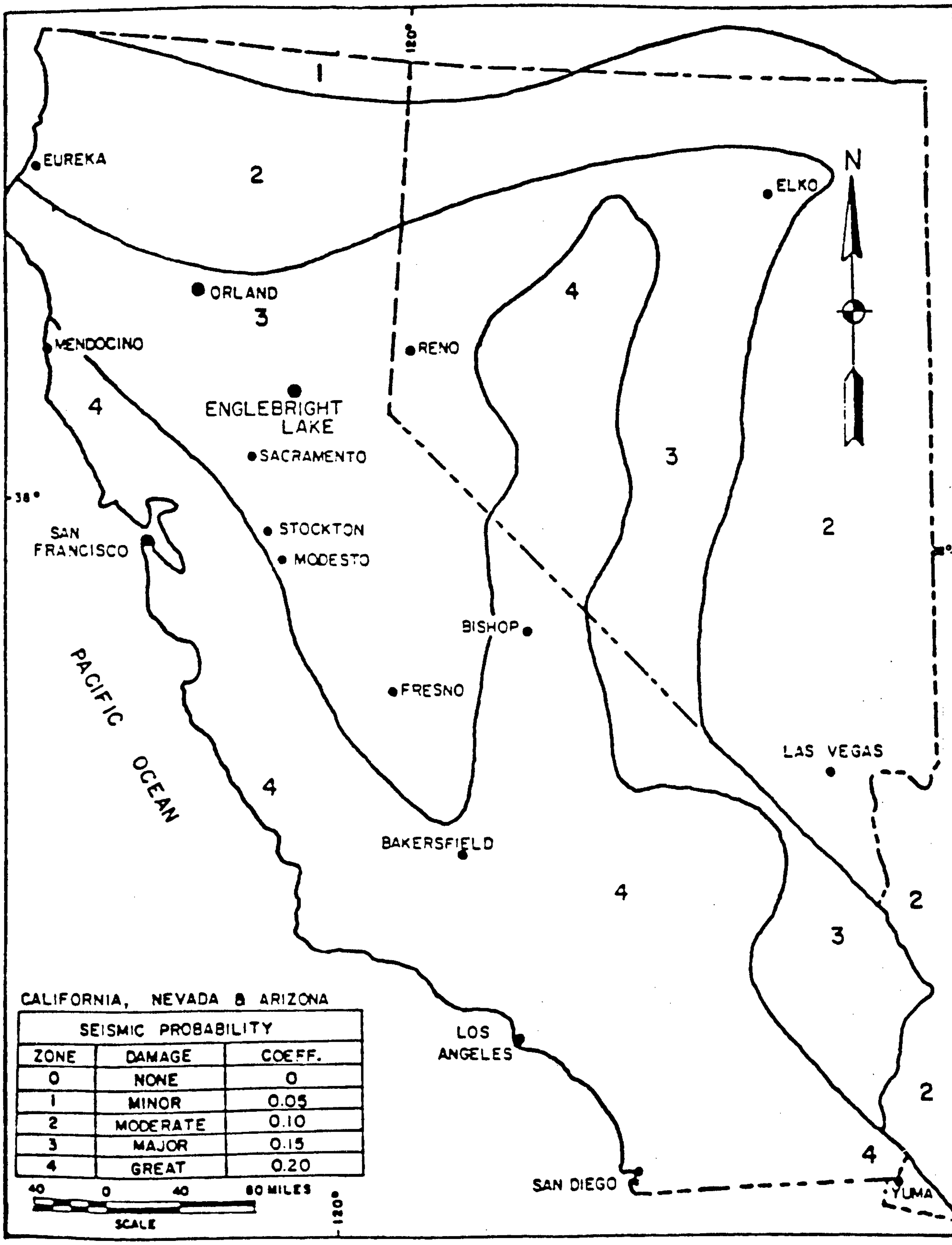
*EFFECTS OF STATIC LOADS ARE EXCLUDED

TABLE 9-7 SUMMARY OF MAXIMUM TENSILE STRESSES ON UPSTREAM FACE, DOWNSTREAM FACE, AND MID-FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1 AND QUAKE 2 EXCITATIONS. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

α	UPSTREAM FACE			DOWNSTREAM FACE			MID-FACE		
	MAXIMUM TENSILE STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	NODAL POINT NUMBER	MAXIMUM TENSILE STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	NODAL POINT NUMBER	MAXIMUM TENSILE STRESS (PSI)	TIME OF OCCURRENCE (SEC.)	NODAL POINT NUMBER
QUAKE 1 η = 0.14									
0.90	1295	4.92	472	846	7.24	149	867	7.26	217
0.75	1128	4.92	472	786	7.24	149	869	7.26	217
QUAKE 2 η = 0.14									
0.90	1791	4.66	303	1397	4.66	149	1404	4.66	217
0.75	1638	4.66	303	1278	4.66	149	1283	4.66	217

TABLE 9-8: SUMMARY OF MAXIMUM TENSILE STRESS LEVELS (ARCH AND CANTILEVER STRESSES) AT THE MOST-STRESSED LOCATIONS EXCEEDED SIX TIMES DURING QUAKE 1 AND QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET). CONSTANT HYSTERETIC DAMPING FACTOR = 0.14, WAVE REFLECTION COEFFICIENT = 0.90.

	MAXIMUM TENSILE STRESS LEVELS EXCEEDED SIX TIMES DURING QUAKE 1				MAXIMUM TENSILE STRESS LEVELS EXCEEDED SIX TIMES DURING QUAKE 2			
	DAM NODAL POINT NUMBER	ARCH STRESS (PSI)	DAM NODAL POINT NUMBER	CANTILEVER STRESS (PSI)	DAM NODAL POINT NUMBER	ARCH STRESS (PSI)	DAM NODAL POINT NUMBER	CANTILEVER STRESS (PSI)
UPSTREAM FACE	304	891	472	949	304	851	484	813
	303	816	484	947	303	792	480	799
	319	694	481	918	305	706	481	791
	305	669	486	909	319	686	476	785
	302	638	476	908	320	595	472	746
	336	585	487	886	336	578	486	727
	337	581	480	804	353	535	487	687
	320	567	465	800	337	530	465	664
	353	548	483	797	302	516	463	603
	338	477	463	789	338	502	483	586
	301	443	473	780	369	475	473	571
	306	392	475	776	370	424	475	553
DOWNSTREAM FACE	21	365	184	299	24	314	184	270
	22	358	195	277	21	304	171	256
	20	358	149	268	22	304	195	242
	23	335	171	268	20	298	149	235
	38	299	158	239	23	293	200	185



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.05
2	MODERATE	0.10
3	MAJOR	0.15
4	GREAT	0.20

FIGURE 1-1

SEISMIC ZONE MAP

ENGLEBRIGHT LAKE

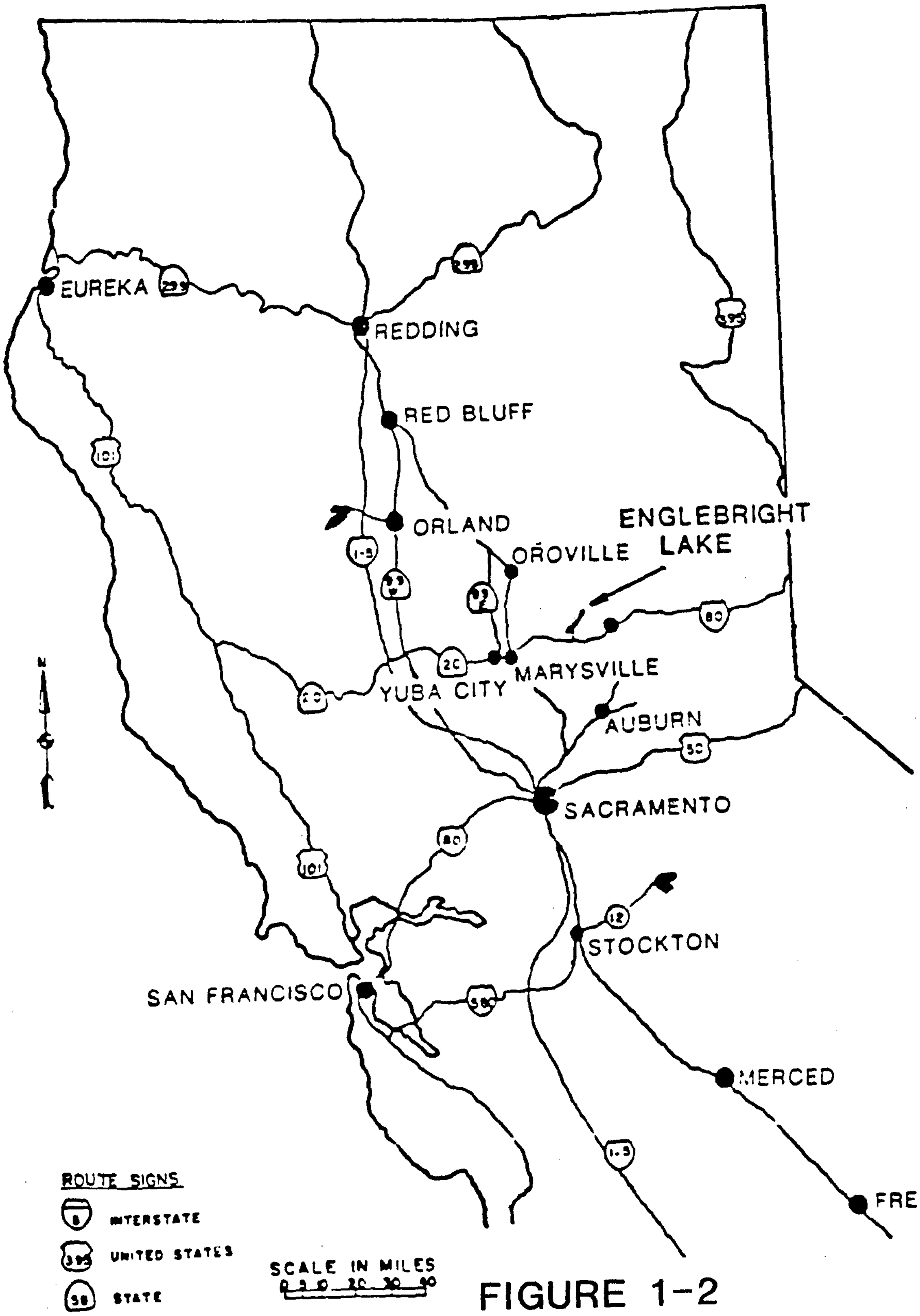
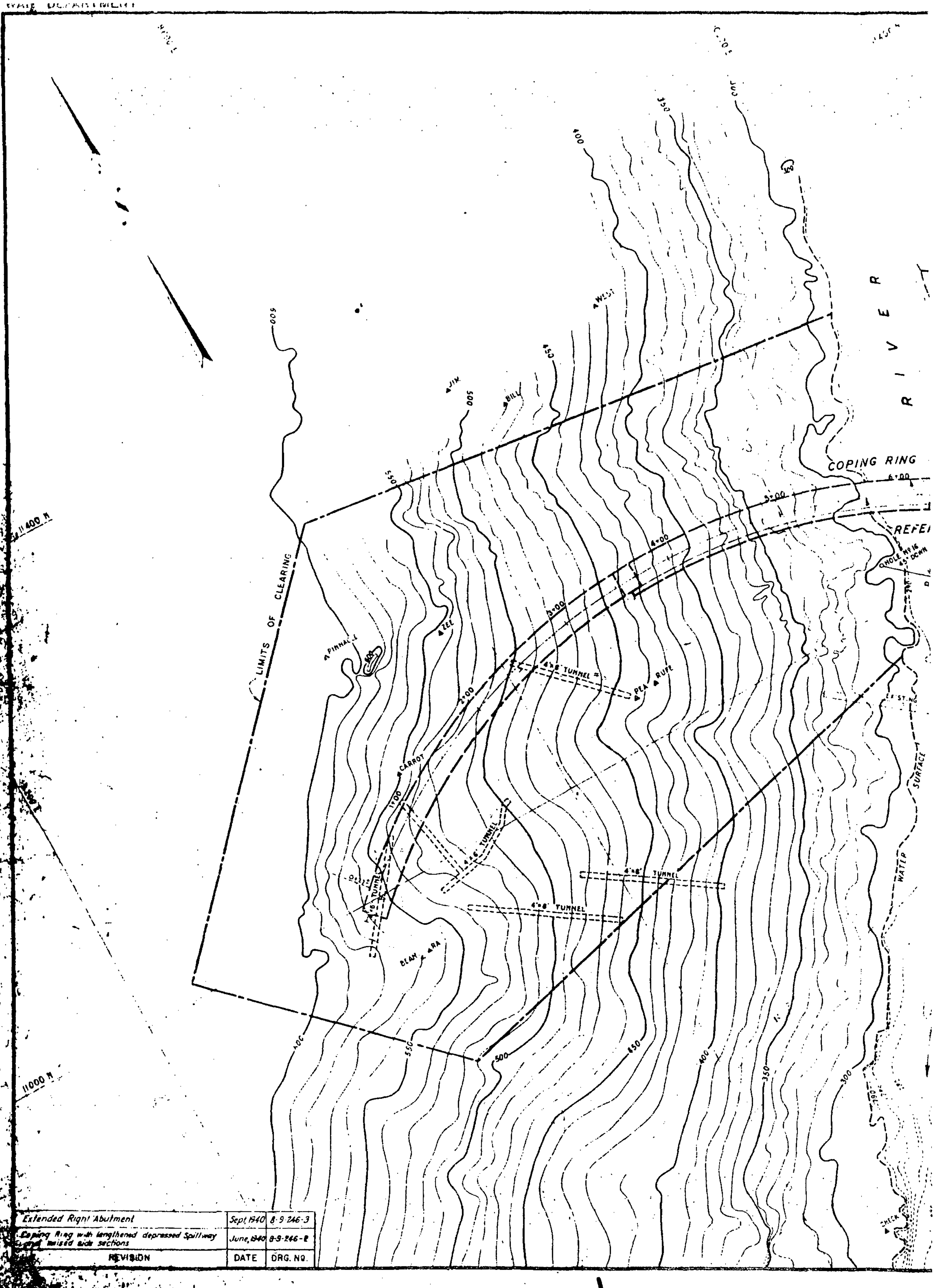
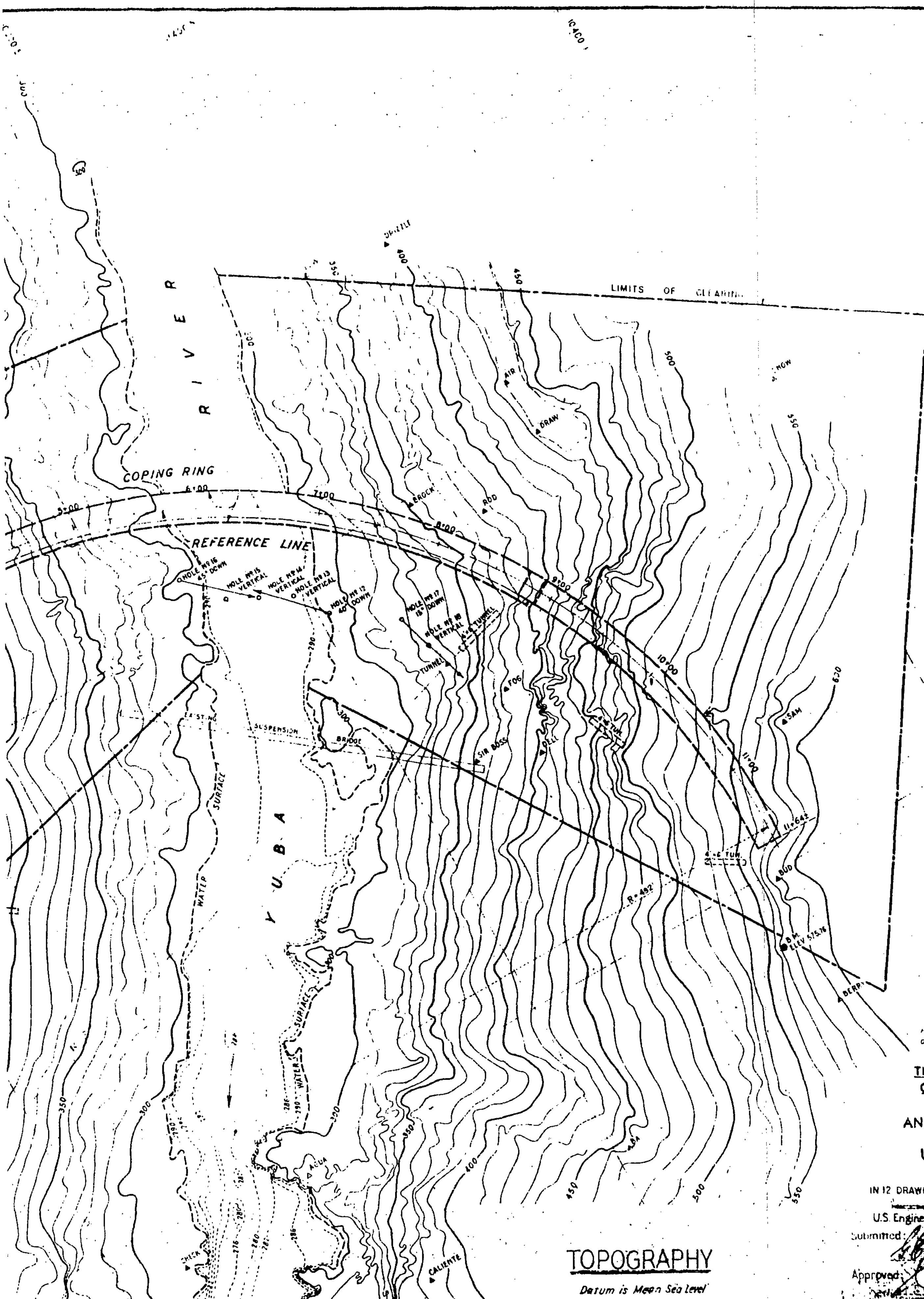


FIGURE 1-2
VICINITY MAP



Extended Right Abutment	Sept 1940	8-9-246-3
Coping Ring with lengthened depressed Spillway and raised side sections	June, 1940	8-9-246-2
REVISION	DATE	DRG. NO.



TOPOGRAPHY

Datum is Mean Sea Level

LEG

LOCATION, BEARING
SUB-SURFACE DIA

FIGU

PI

This drawing is one of tw
No. 1-04 of Circular Proposa
and forms a part thereof.

TO ACCOMPANY
THE ARUNDEL COR
CONTRACTORS, CO

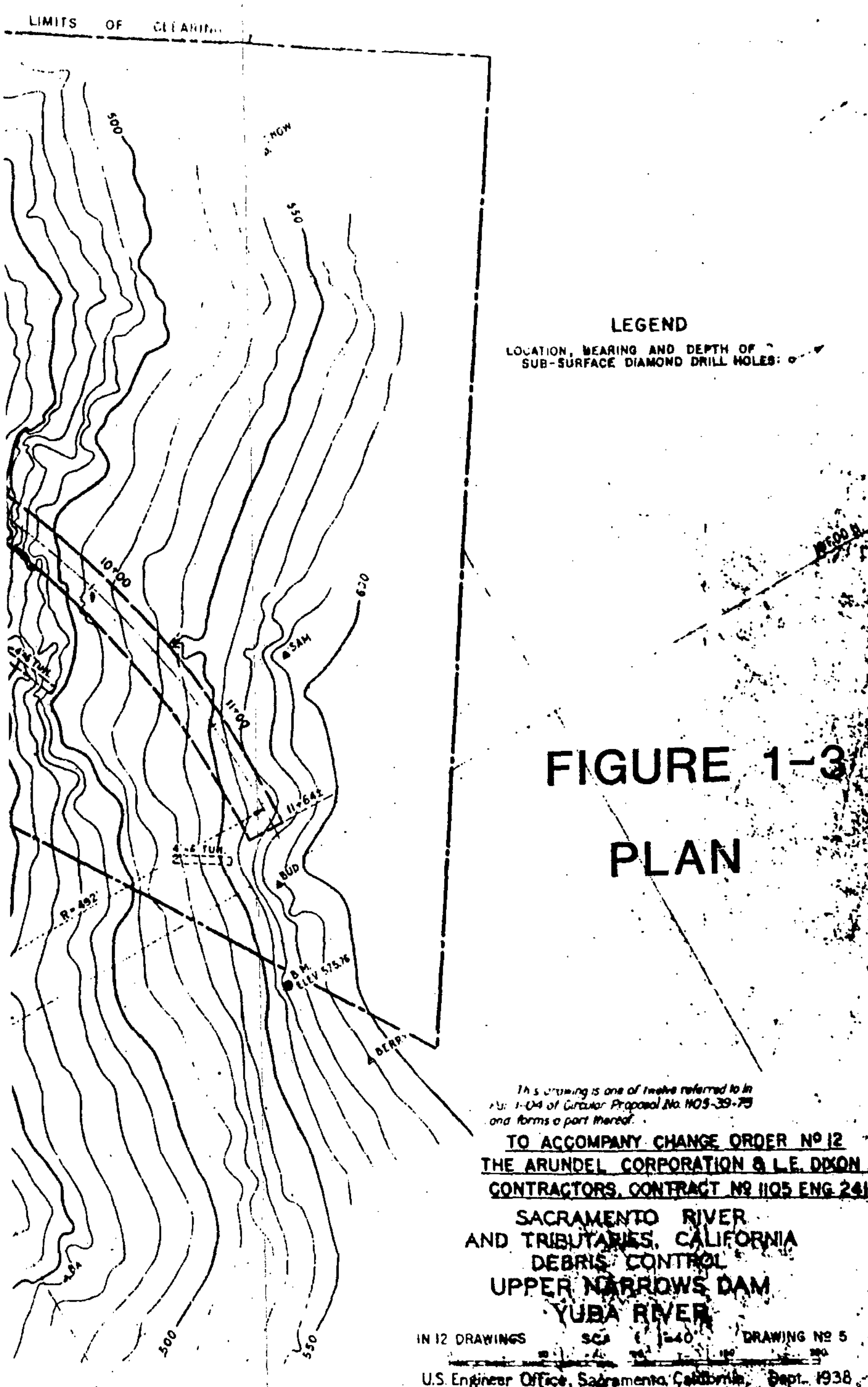
SACRAMENTO
AND TRIBUTARIES
DEBRIS CO
UPPER NARR
YUBA RI

IN 12 DRAWINGS SCA

U.S. Engineer Office, Sacrament

Submitted:

Approved:



LEGEND

LOCATION, BEARING AND DEPTH OF SUB-SURFACE DIAMOND DRILL HOLES:

FIGURE 1-3

PLAN

This drawing is one of twelve referred to in ... 1-04 of Circular Proposal No. 1105-39-75 and forms a part thereof.

TO ACCOMPANY CHANGE ORDER Nº 12
THE ARUNDEL CORPORATION & L.E. DIXON CO.
CONTRACTORS, CONTRACT Nº 1105 ENG 2411

SACRAMENTO RIVER
AND TRIBUTARIES, CALIFORNIA
DEBRIS CONTROL
UPPER NARROWS DAM
YUBA RIVER

IN 12 DRAWINGS SCALE 1"=40' DRAWING Nº 5

U.S. Engineer Office, Sacramento, California, Sept., 1938

Submitted: Approval Recommended:

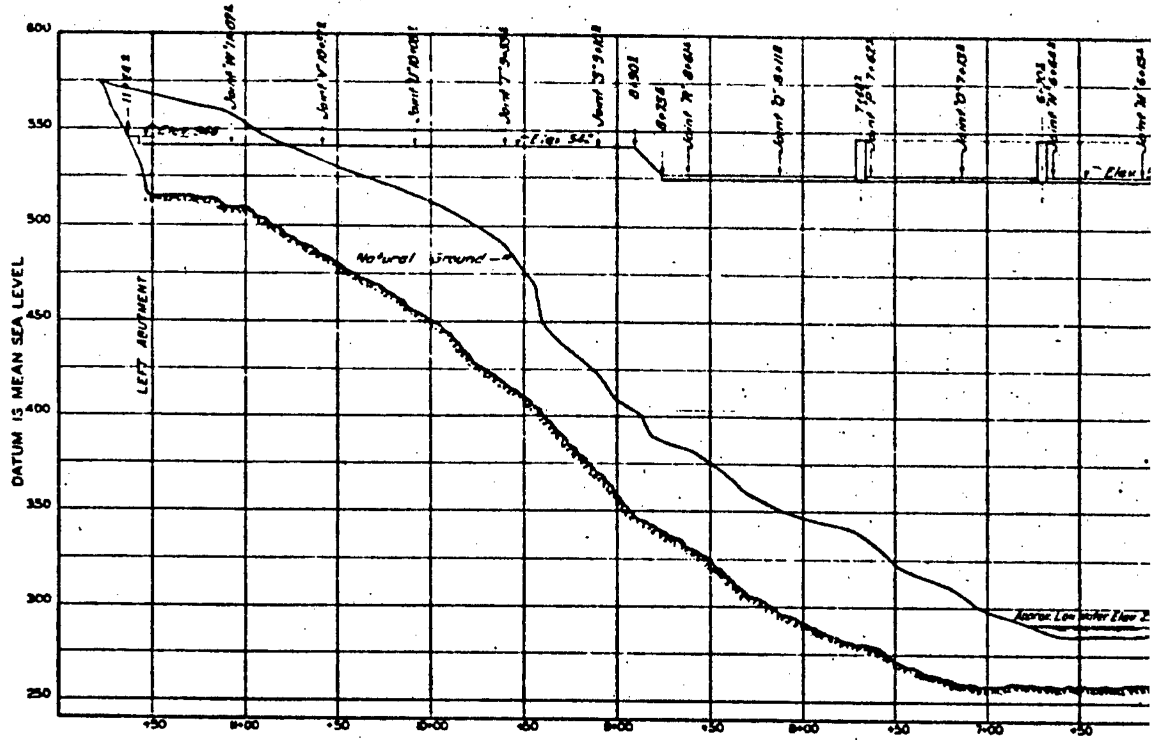
Approved:

...POGRAPHY

...tum is Mean Sea Level

3

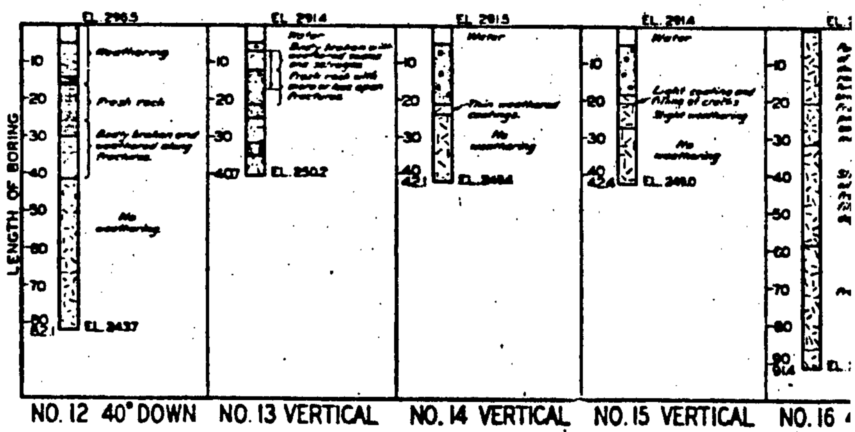
WAR DEPARTMENT



DEVELOPED PROF

LEGEND

- Overburden
- Sand Gravel and Boulders
- Quartz Diorite
- Uralite Diabase
- Meta-Basalt
- Metamorphosed Breccia
- Lost Water



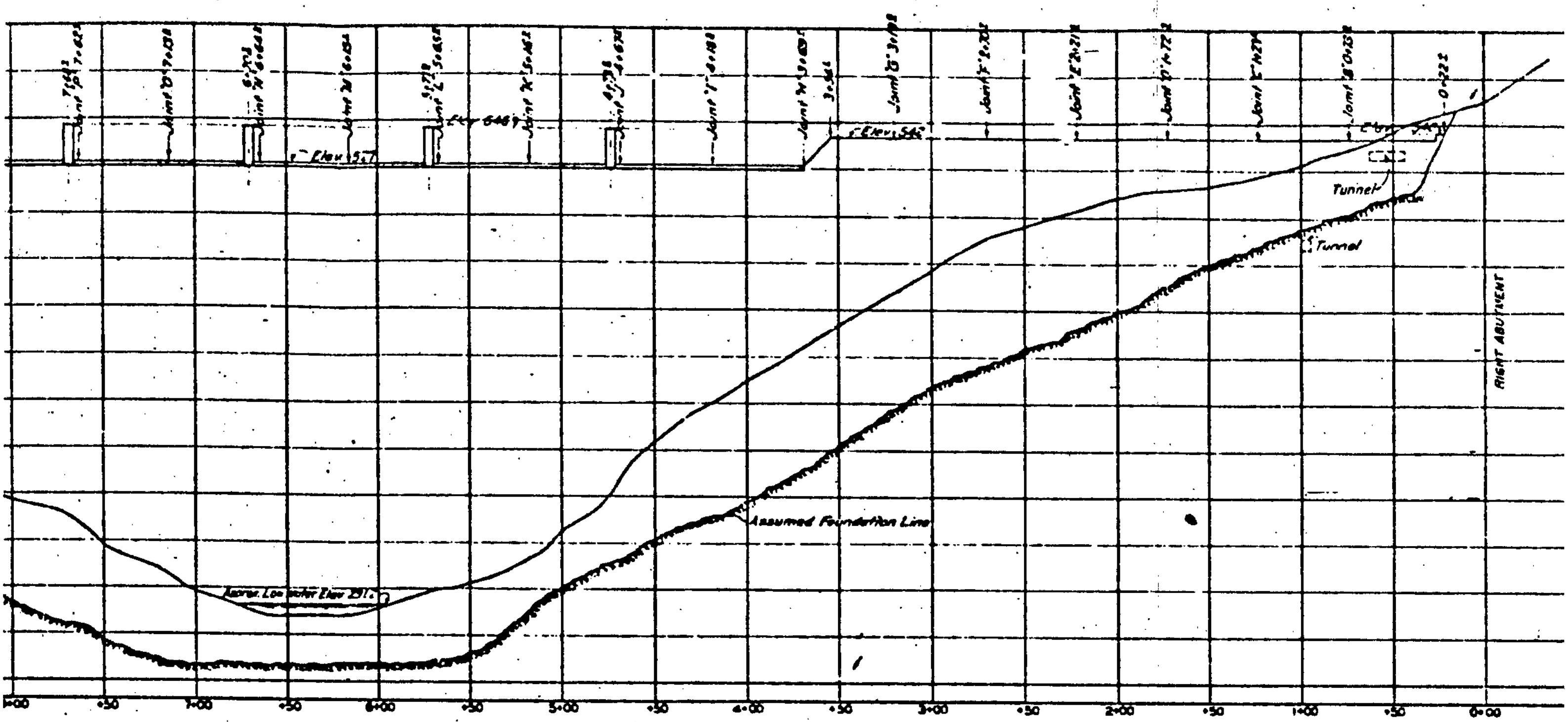
BORINGS

DATA OBTAINED FROM DIAMOND DRILL BOR

NOTE:
Datum is mean sea level

DEVELOPED P

Increased length of depressed spillway, revised side sections, aeration piers	[illegible]	[illegible]
Right Abutment Retaining Wall	5-29-40	[illegible]
Aeration Piers moved and Depressed Section increased	[illegible]	[illegible]
Low water and foundation line references added	[illegible]	[illegible]
REVISION	DATE	DRG NO

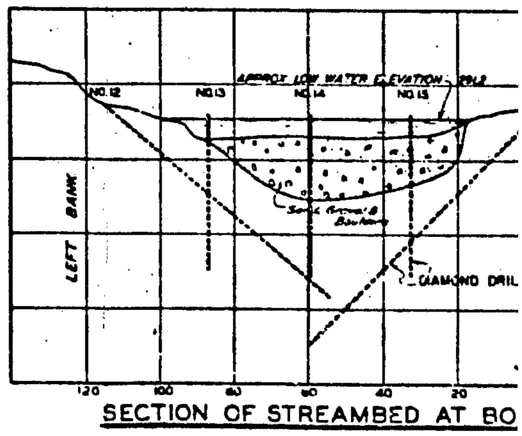


DEVELOPED PROFILE LOOKING DOWNSTREAM

Scale: 1"=40'

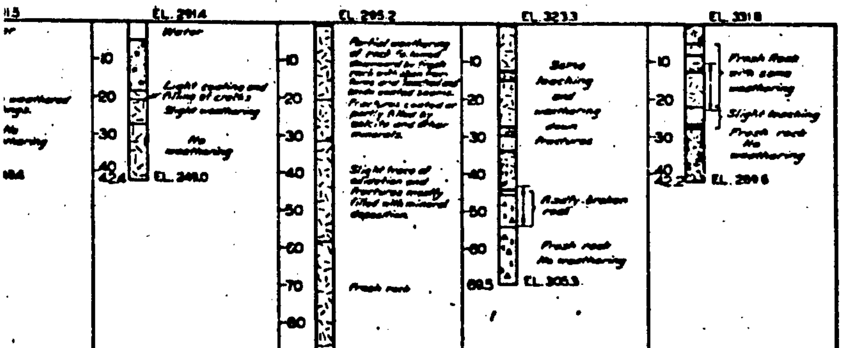
FIGUR

U.S. EL



SECTION OF STREAMBED AT BO

Scale 1"=20'



BORINGS

OBTAINED FROM DIAMOND DRILL BORINGS

DEVELOPED PROFILE AND BORINGS

This drawing is one of twelve referred to in Par. 1-04 of Circular Proposal No. 1105-3C-75 and forms a part thereof.

REFERENCE DRAWINGS:

Topography, Tunnels and Borings: Drawing No. 5, 8-9-296-2
Plan of Arch Rings and Excavation: Drawing No. 1, 8-9-296-2
Contraction Joints: Drawing Nos. 2, 3, & 4, 8-9-296-2
Aeration piers and coping details: Drawing No. 1, 8-9-317

TO ACCOMPANY CHANGE ORDER NO 12
THE ARUNDEL CORPORATION & L.E. DIXON CO.
CONTRACTORS, CONTRACT NO 1105 ENG 2411

SACRAM
AND TRIBUT
DEBRIS
UPPER N
YUB

IN 12 DRAWINGS SCALE

U.S. Engineer Office, Sacr

Submitted:

Approved:

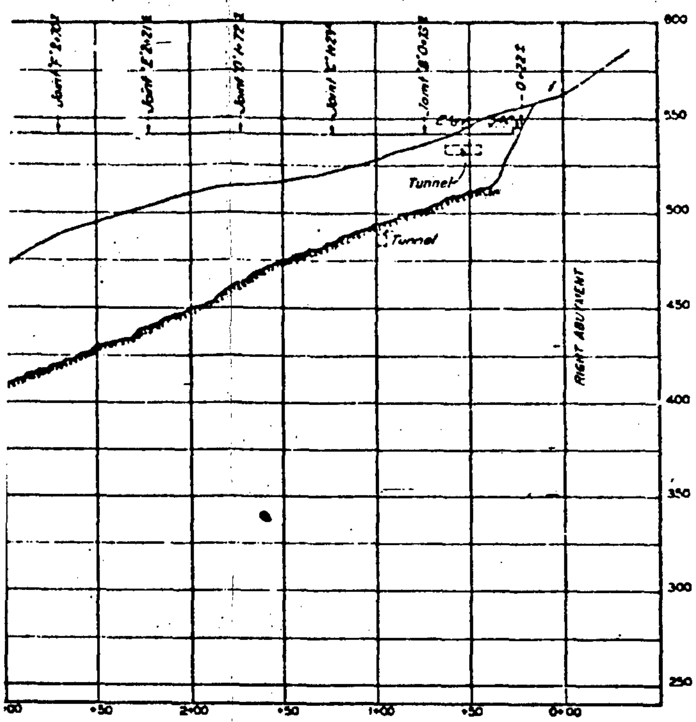
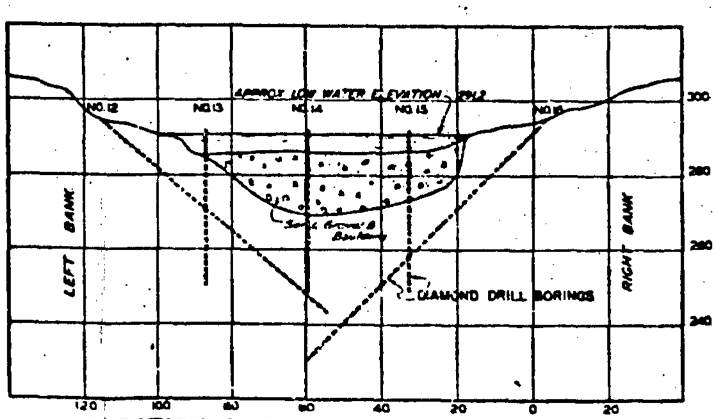


FIGURE 1-4

U.S. ELEVATION



SECTION OF STREAMBED AT BORINGS

Scale 1"= 20'

This drawing is one of twelve referred to in Par. 1-04 of Circular Proposal No. 1105-30-75 and forms a part thereof.

SACRAMENTO RIVER
AND TRIBUTARIES, CALIFORNIA
DEBRIS CONTROL
UPPER NARROWS DAM
YUBA RIVER

REFERENCE DRAWINGS:

Topography, Tunnels and Borings: Drawing No. 5, 8-9-296-2
Plan of Arch Rings and Excavation: Drawing No. 1, 8-9-296-2
Contraction Joints: Drawing Nos. 2, 3 & 4, 8-9-296-2
Aeration piers and coping details: Drawing No. 1, 8-9-317

IN 12 DRAWINGS SCALE AS SHOWN DRAWING NO. 7

U.S. Engineer Office, Sacramento, California, Sept. 1938

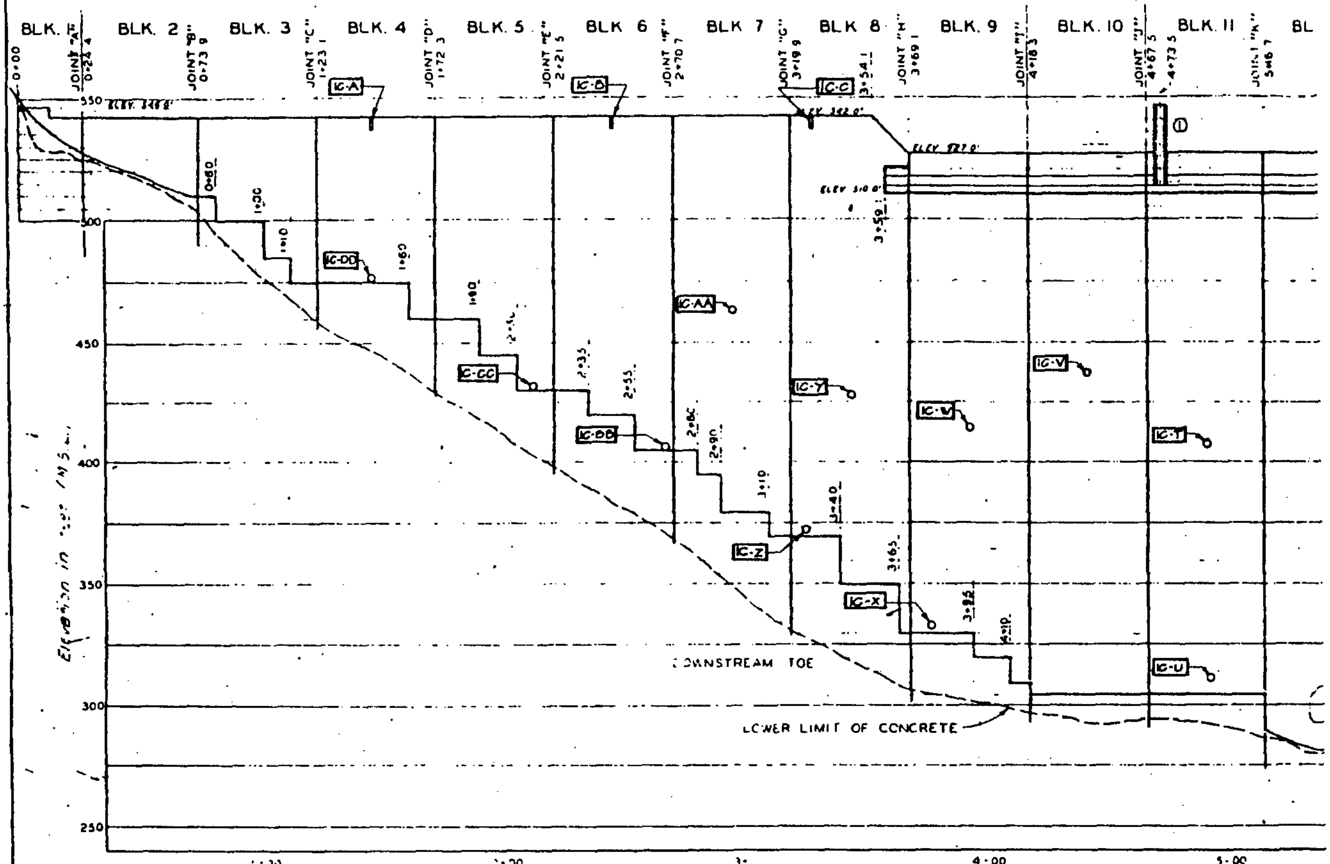
Submitted: Approval Recommended:

Approved:

TO ACCOMPANY CHANGE ORDER NO 12
THE ARUNDEL CORPORATION & L.E. DIXON CO.
CONTRACTORS, CONTRACT NO 1105 ENG 2411

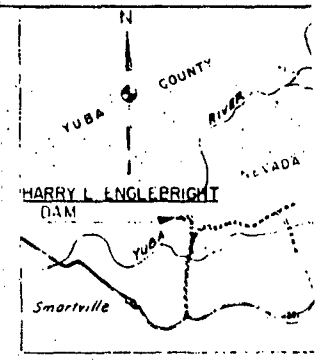
8 9 296

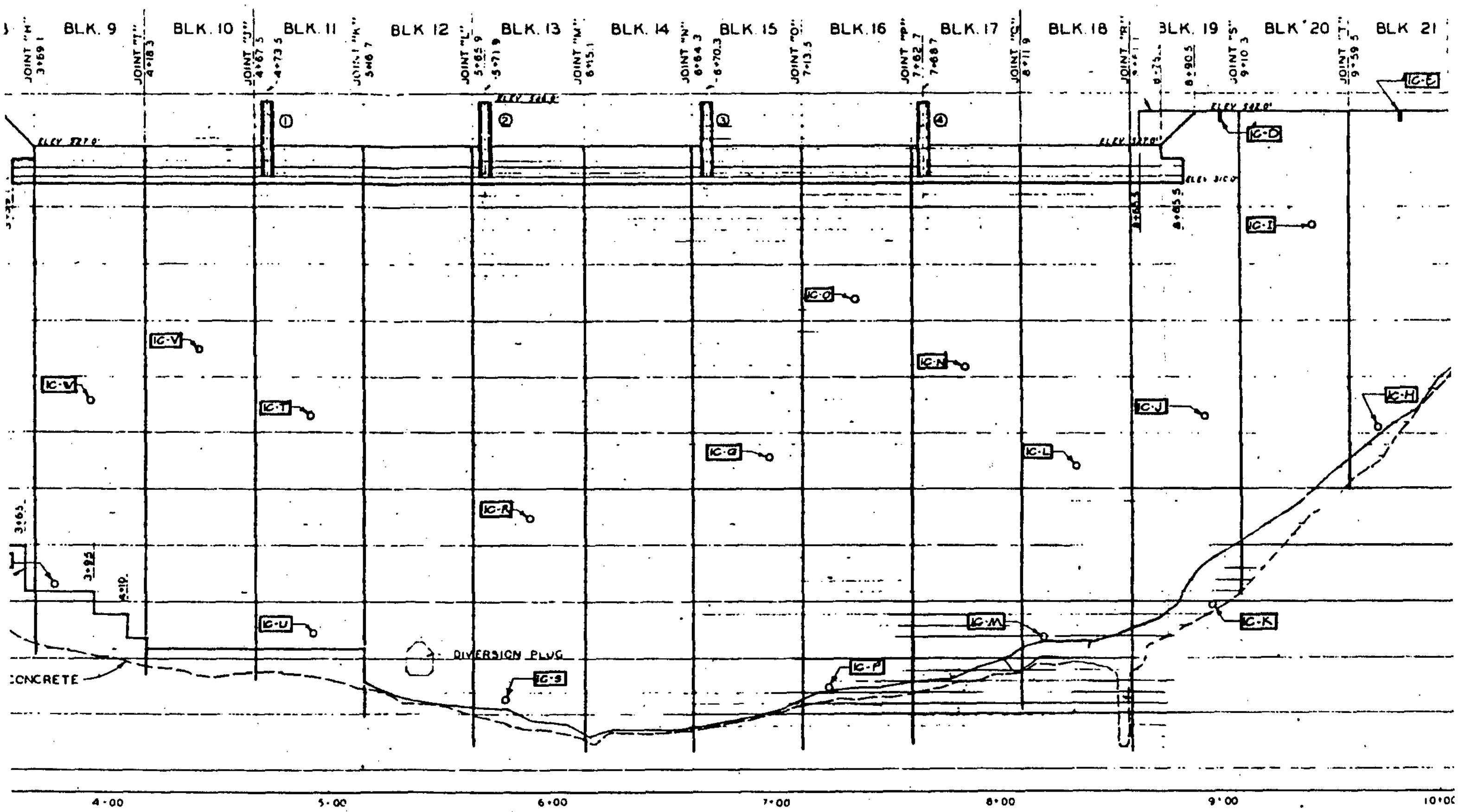
3



BLK. 2
BLK. 3
BLK. 4
BLK. 5
BLK. 6
BLK. 7
BLK. 8
BLK. 9
BLK. 10
BLK. 11
JOINT "B" 0+73.9
JOINT "C" 1+23.1
JOINT "D" 1+72.3
JOINT "E" 2+21.5
JOINT "F" 2+70.7
JOINT "G" 3+19.9
JOINT "H" 3+69.1
IC-A
IC-B
IC-C
IC-DD
IC-AA
IC-CC
IC-BB
IC-Y
IC-W
IC-V
IC-T
IC-Z
IC-X
IC-U
DOWNSTREAM TOE
LOWER LIMIT OF CONCRETE
550
500
450
400
350
300
250
4+00
5+00

DEVEL





DEVELOPED PROFILE LOOKING UPSTREAM

SCALE 1" = 30'-0"

GR

D.S

NOTE:
1. 30' DEEP FOUNDATION HOLES TH-A THROUGH TH-D SHOWN ON SHEET 1.
2. CONCRETE CORE DRILL SITES IC-A TO IC-DD

REVISION	DATE

DESIGNED: J. HESS
DRAWN: B. SIMPSON
CHECKED: R. TREAT
SUBMITTED: Justin B. Mo[illegible]

2

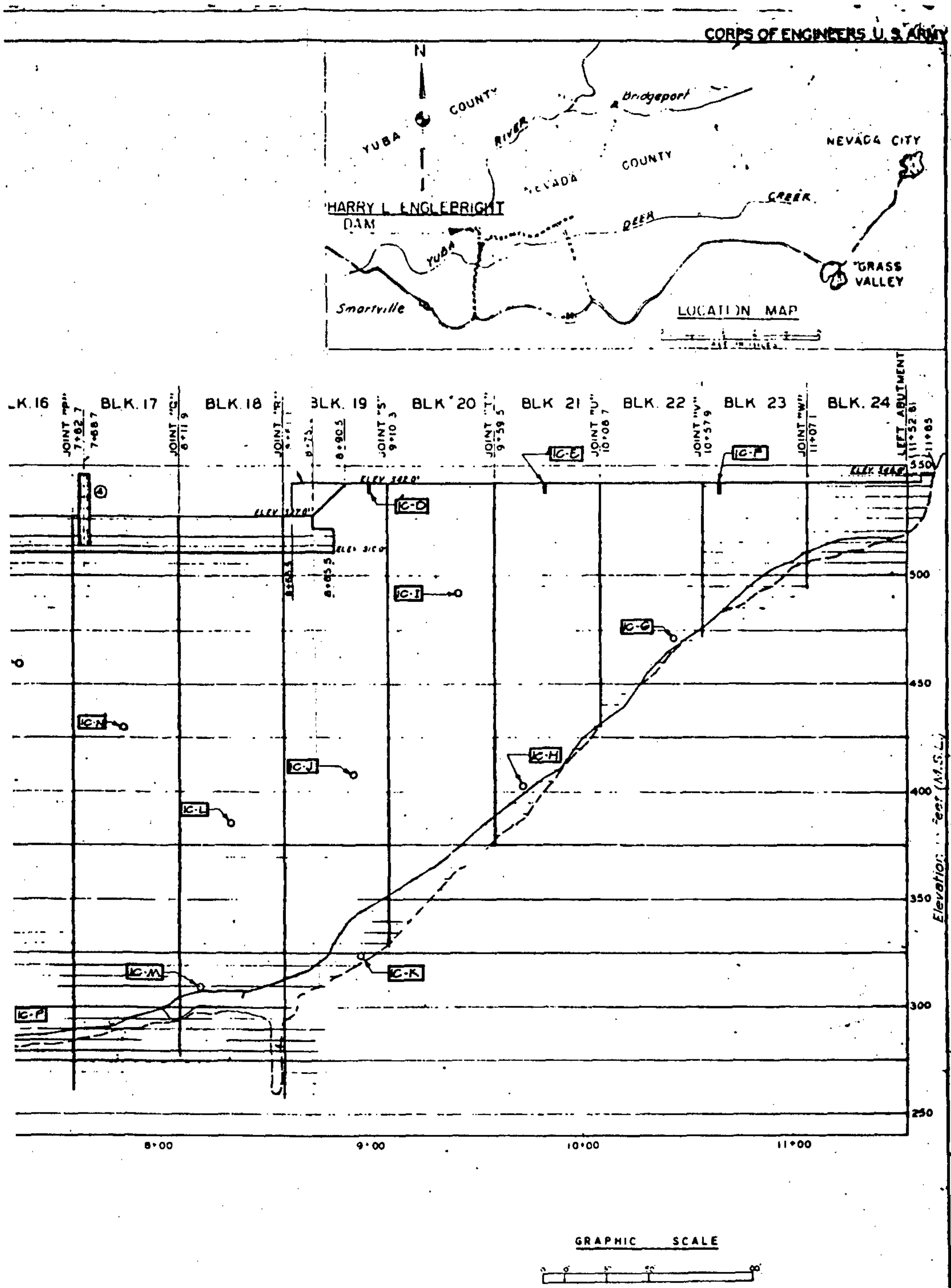


FIGURE 1–5

D.S. ELEVATION

...TION HOLES TH-A
...HOWN ON SHEET 1.

· DRILL SITES IC-A TO IC-DD

REVISION	DATE	DESCRIPTION	BY	BY

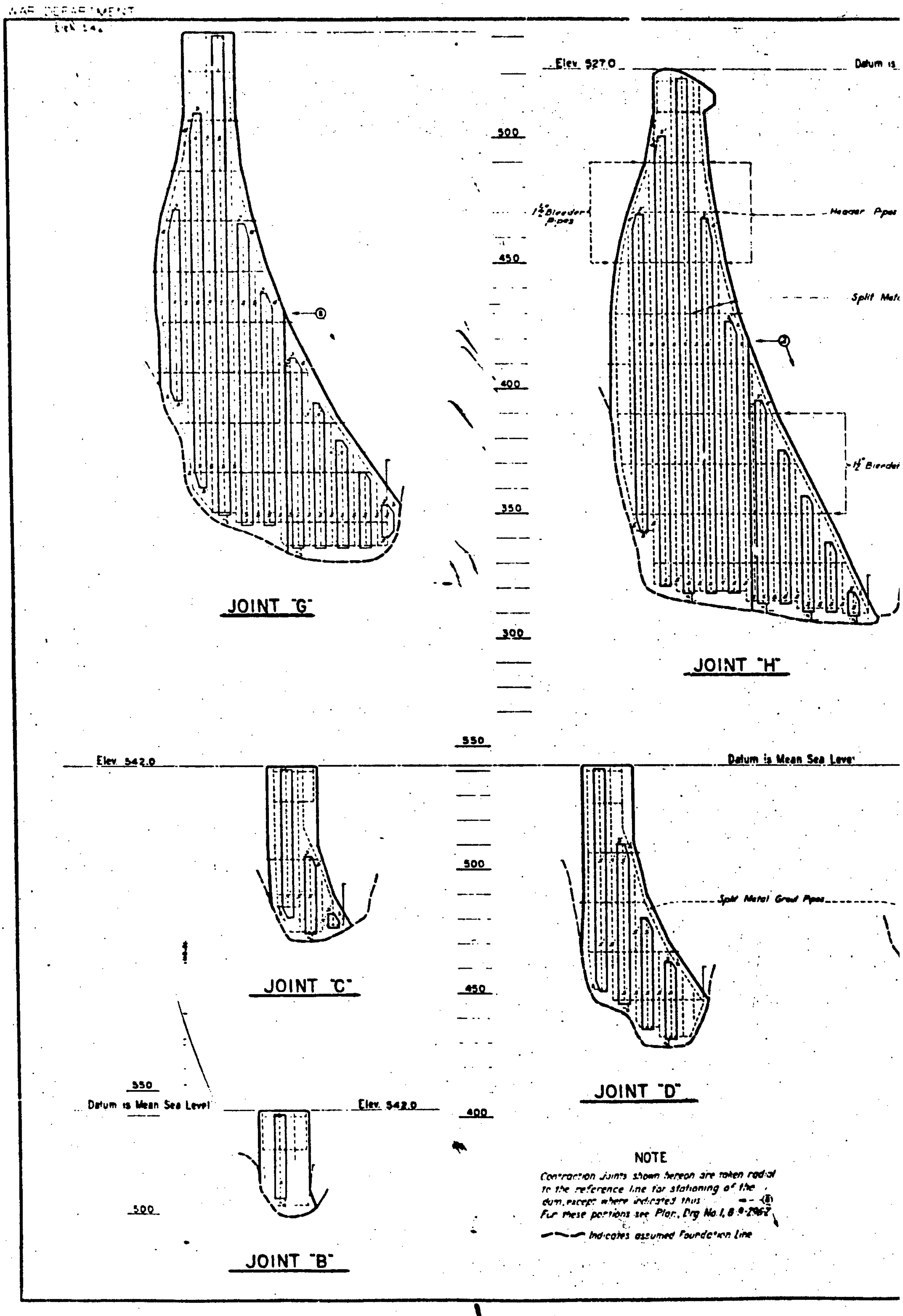
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

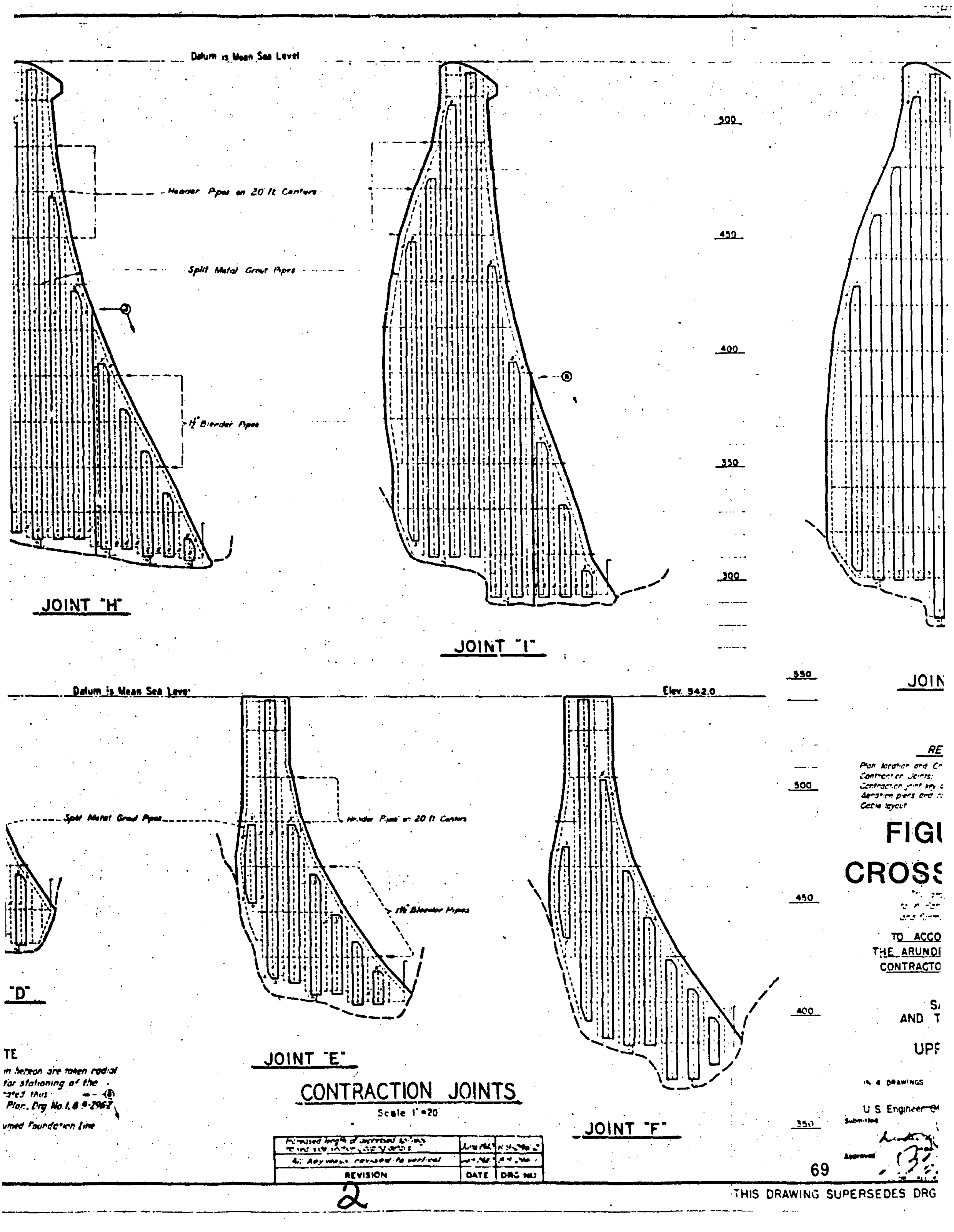
DESIGNED: J. HESS
DRAWN: B. SIMPSON
CHECKED: R. TREAT

ENGLEBRIGHT AND PINE FLAT DAMS, CALIFORNIA
CONCRETE CORE DRILLING,
ROCK CORE DRILLING AND TESTING
ENGLEBRIGHT DAM
ELEVATION

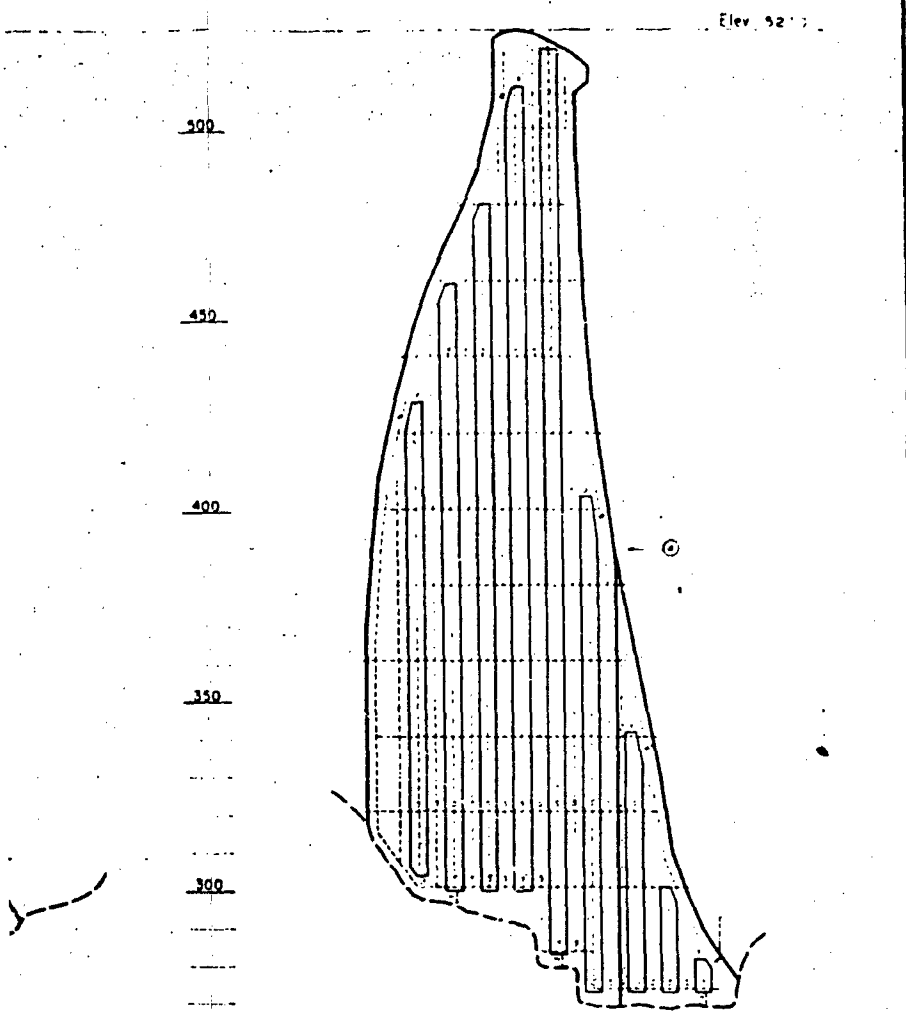
SUBMITTED: Justin B Moore
DATE APPROVED: 19 Feb 82
SCALE: AS SHOWN
SPEC. No.
SHEET: 2
FILE No. 8-1-844

3

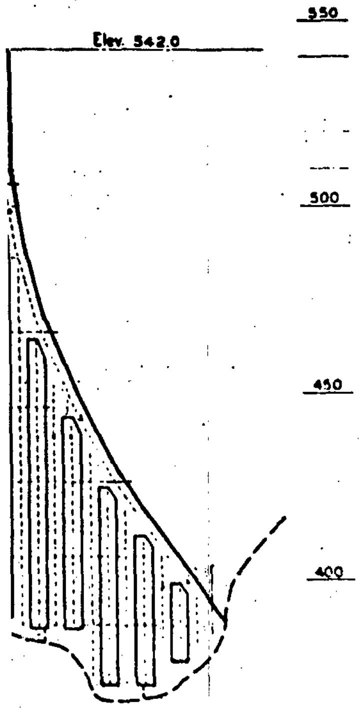

WAR DEPARTMENT
Elev. 527.0
Datum is
500
1½" Bleeder Pipes
Header Pipes
450
Split Meta
400
1½" Bleeder
350
300
JOINT "G"
JOINT "H"
550
Elev. 542.0
Datum is Mean Sea Level
500
Split Metal Grout Pipes
450
JOINT "C"
JOINT "D"
550
Datum is Mean Sea Level
Elev. 542.0
400
500
JOINT "B"
NOTE
Contraction Joints shown hereon are taken radial to the reference line for stationing of the dam, except where indicated thus
For these portions see Plan, Drg No. 1, 8-9-2962
Indicates assumed Foundation Line



Datum is Mean Sea Level
Header Pipes on 20 ft Centers
Split Metal Grout Pipes
1½" Bleeder Pipes
JOINT "H"
JOINT "I"
500
450
400
350
300
550
Datum is Mean Sea Level
Elev. 542.0
Split Metal Grout Pipes
Header Pipes on 20 ft Centers
1½" Bleeder Pipes
"D"
JOINT "E"
JOINT "F"
CONTRACTION JOINTS
Scale 1"=20'
REVISION
DATE
DRG NO
JOIN
RE
FIGU
CROSS
TO ACCO
THE ARUNDI
CONTRACTO
AND T
UPF
U S Engineer
Submitted
Approved
69
THIS DRAWING SUPERSEDES DRG
2



JOINT "J"



NT "F"

REFERENCE DRAWINGS

Plan location and Crown Section	[illegible]
Contraction Joints:	[illegible]
Contraction joint key and grouting details	[illegible]
Aeration piers and piping details	[illegible]
Cable layout	[illegible]

FIGURE 1–6
CROSS SECTIONS

TO ACCOMPANY CHANGE ORDER Nº 12
THE ARUNDEL CORPORATION & L.E. DIXON CO.
CONTRACTORS, CONTRACT Nº 105 ENG 24

SACRAMENTO RIVER
AND TRIBUTARIES, CALIFORNIA
DEBRIS CONTROL
UPPER NARROWS DAM
YUBA RIVER

IN 4 DRAWINGS SCALE AS SHOWN DRAWING NO.

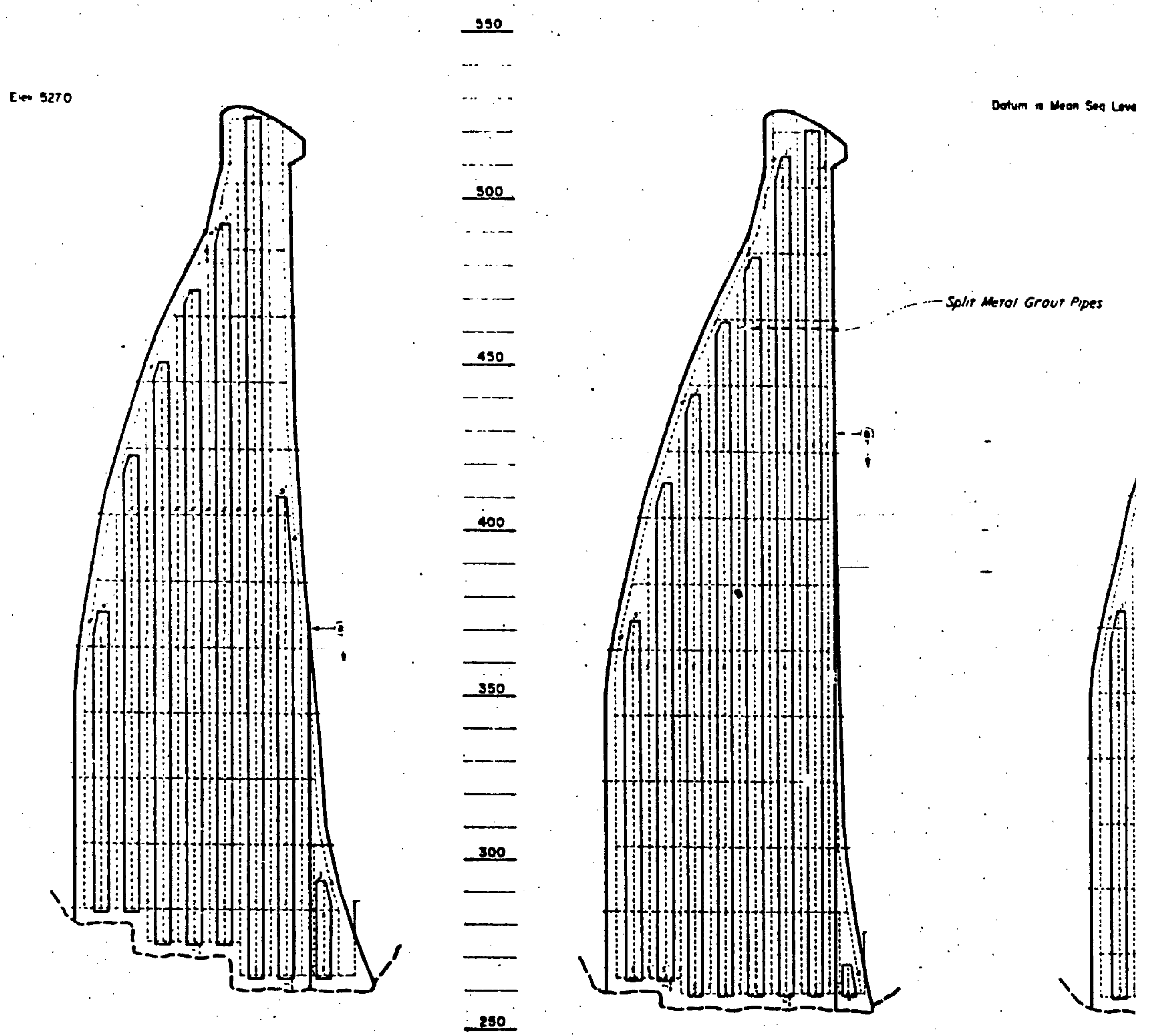
U.S. Engineer Office Sacramento, California Dec. 1939

Submitted Approval Recommended

Approved

THIS DRAWING SUPERSEDES DRG. Nº 8, 8-9-246-2, DATED AUG

WAR DEPARTMENT



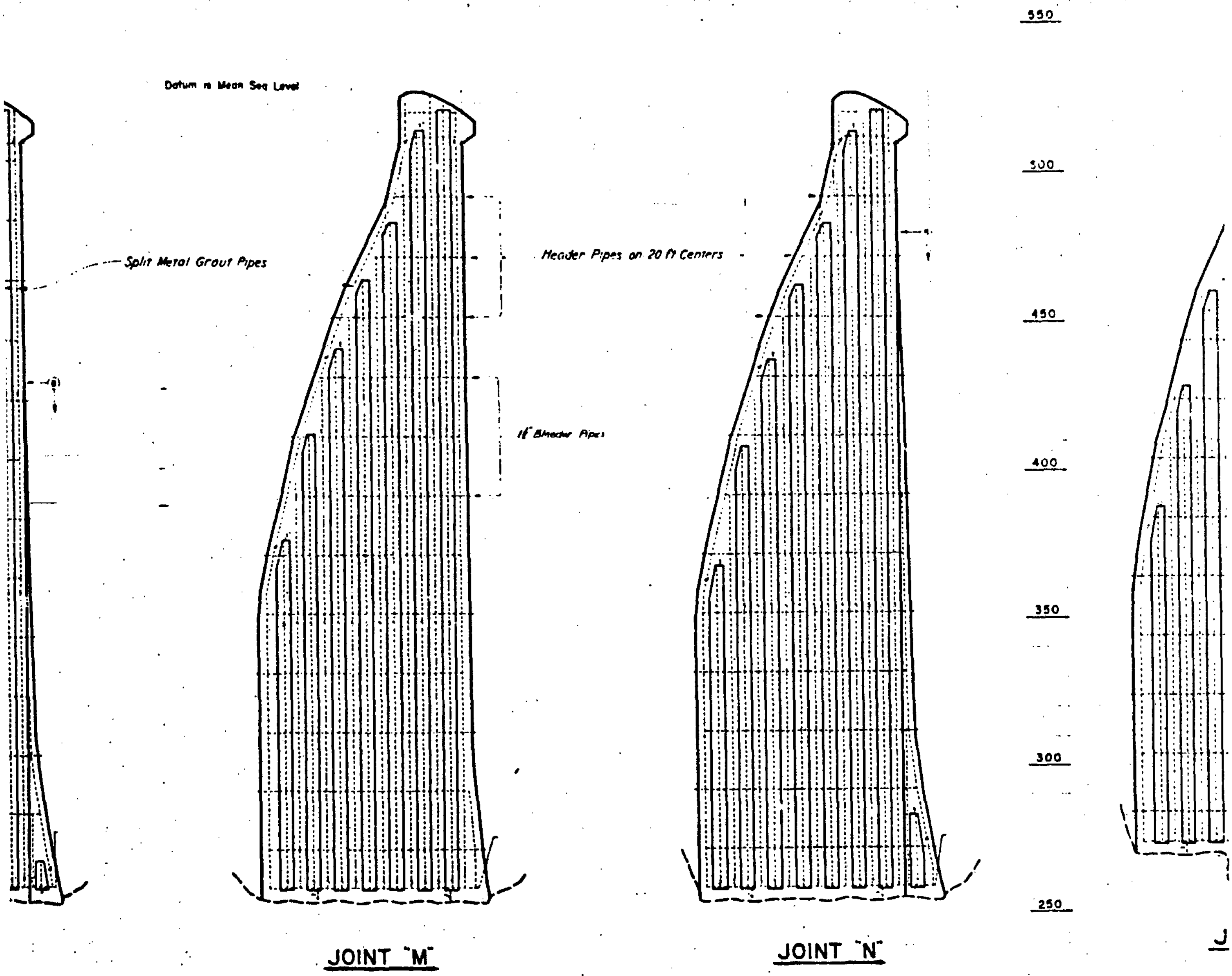
JOINT "K"

JOINT "L"

NOTE

[illegible] shown hereon are taken radial [illegible] reference line for stationing of [illegible] except where indicated thus [illegible] for these portions see Plan Drg [illegible] 2

[illegible] Assumed Fan line



REFERENCE DRAWINGS

Plan location and crown section	Drg N° 1, 8-9-296 2
Contraction Joints	Drg Nos 3 & 4, 8-9-296-2
Contraction joint key and grouting details	Drg No 4, 8-9-296-2
Aeration piers and coping details	Drg No 1, 8-9-317
Cable layout	Drg No 1, 8-9-299-2

TO ACCOMP
THE ARUNDEL
CONTRACTORS,

SAC
AND TRI
DI
UPPE

FIGURE 1-7

CROSS SECTIONS

CONTRACTION JOINTS

Scale 1" = 20'

IN 4 DRAWINGS

U.S. Engineer Offic

Submitted

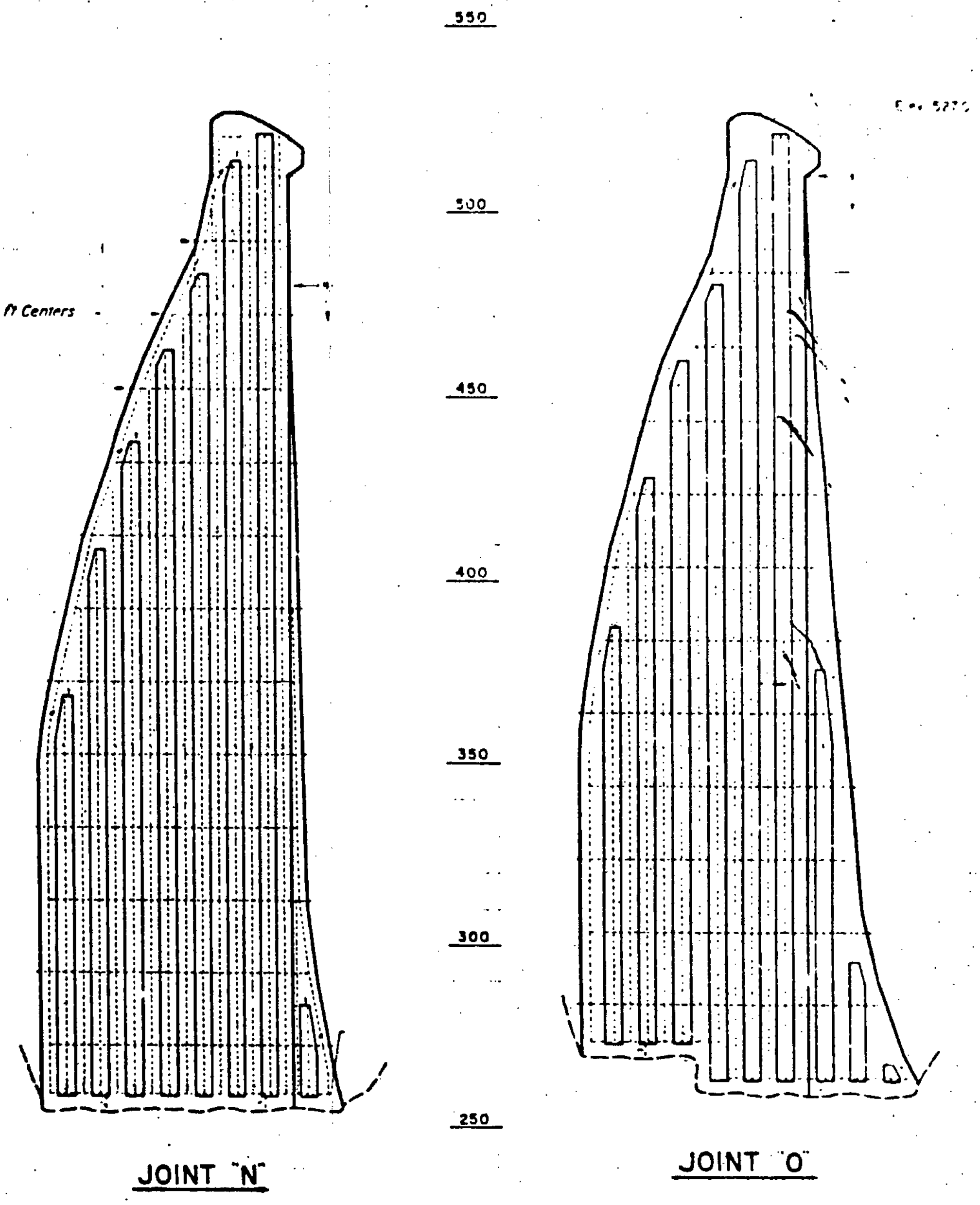
Approved

[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
REVISION	DATE	DRG. N°

70

THIS DRAWING SUPERSEDES DRG N

CORPS OF ENGINEERS, U. S. ARMY



REFERENCE DRAWINGS

location and crown section	Drg Nº 1, 8-9-296-2
action Joints	Drg Nos 3 & 4, 8-9-296-2
action joint key and grouting details	Drg No 4, 8-9-296-2
on piers and coping details	Drg No 1, 8-9-317
layout	Drg No 2, 8-9-299-2

FIGURE 1-7

CROSS SECTIONS

TO ACCOMPANY CHANGE ORDER Nº 12
THE ARUNDEL CORPORATION & L.E. DIXON CO.
CONTRACTORS, CONTRACT Nº 1105 ENG 2411

SACRAMENTO RIVER
AND TRIBUTARIES, CALIFORNIA
DEBRIS CONTROL
UPPER NARROWS DAM
YUBA RIVER

IN 4 DRAWINGS SCALE AS SHOWN DRAWING No.

U.S. Engineer Office, Sacramento California, Dec 1939

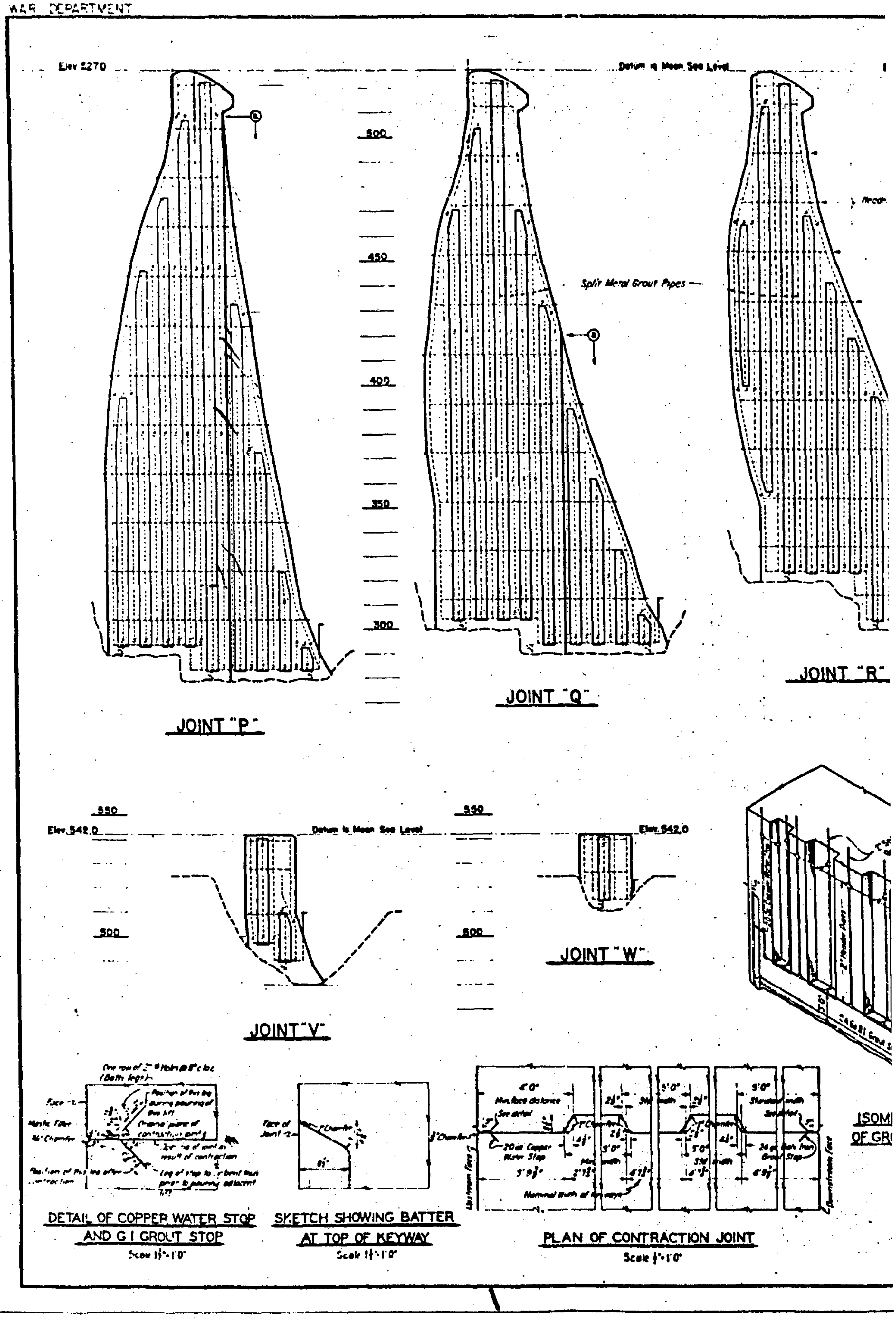
Submitted

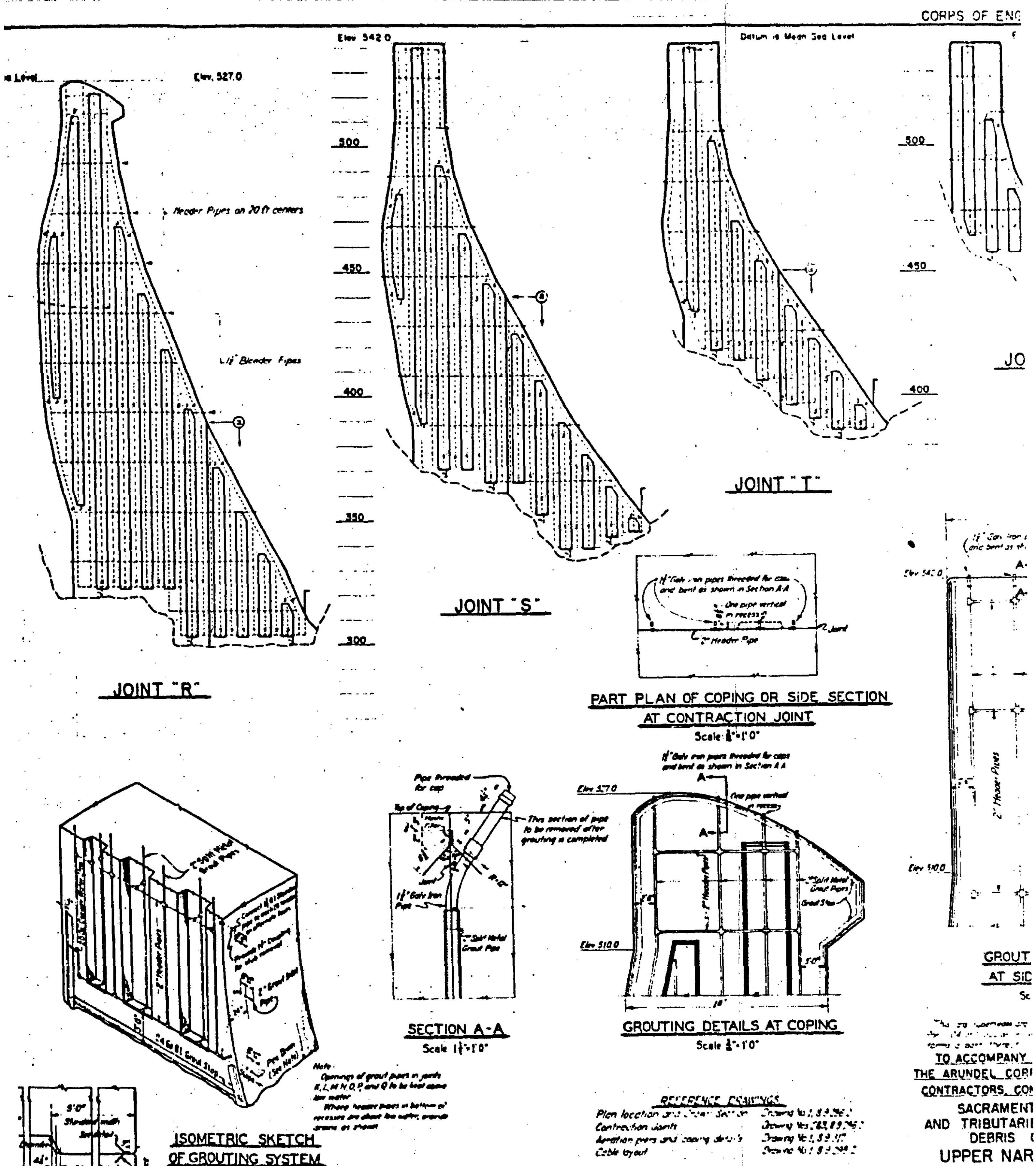
Approved

REVISION	DATE	DRG. No

70

THIS DRAWING SUPERSEDES DRG Nº 9, 8-9-246-2, DATED AUG


Elev 527.0
Datum is Mean Sea Level
500
450
400
350
300
Split Metal Grout Pipes
JOINT "P"
JOINT "Q"
JOINT "R"
550
Elev 542.0
Datum is Mean Sea Level
500
JOINT "V"
JOINT "W"
DETAIL OF COPPER WATER STOP
AND G I GROUT STOP
Scale 1½"=1'0"
SKETCH SHOWING BATTER
AT TOP OF KEYWAY
Scale 1½"=1'0"
PLAN OF CONTRACTION JOINT
Scale ½"=1'0"
Upstream Face
Downstream Face
20 oz Copper Water Stop
Nominal Width of Keyways
ISOM
OF GR



Note: Openings of grout pipes in joints K, L, M, N, O, P and Q to be kept above low water. Where header pipes in bottom of recesses are above low water, arrange drains as shown.

REFERENCE DRAWINGS

Plan location and Cross Section	Drawing No. 1, 8.9.296.1
Contraction Joints	Drawing Nos. 283, 8.9.296.2
Aeration piers and coping details	Drawing No. 1, 8.9.117
Cable layout	Drawing No. 1, 8.9.298.2

FIGURE 1-8 CROSS SECTIONS

CONTRACTION JOINTS

Scale 1" = 20'

Note: Contraction Joints shown hereon are taken radial to the reference line for staking of the dam except where indicated thus. For these portions see Plan Drg. No. 1, 8.9.296.2

— — — Indicates Assumed Foundation Line

REVISION	DATE	DRG. NO.
Increased length of depressed sections, added new sections, coping and side section details	June 1942	8-9-296-2
All keyways revised to vertical		8-9-296

TO ACCOMPANY THE ARUNDEL CORP. CONTRACTORS, CO.

SACRAMENTO AND TRIBUTARIES DEBRIS C

UPPER NAR

YUBA R

IN 4 DRAWINGS SCALE AS

U.S. Engineer Office, Sacramento

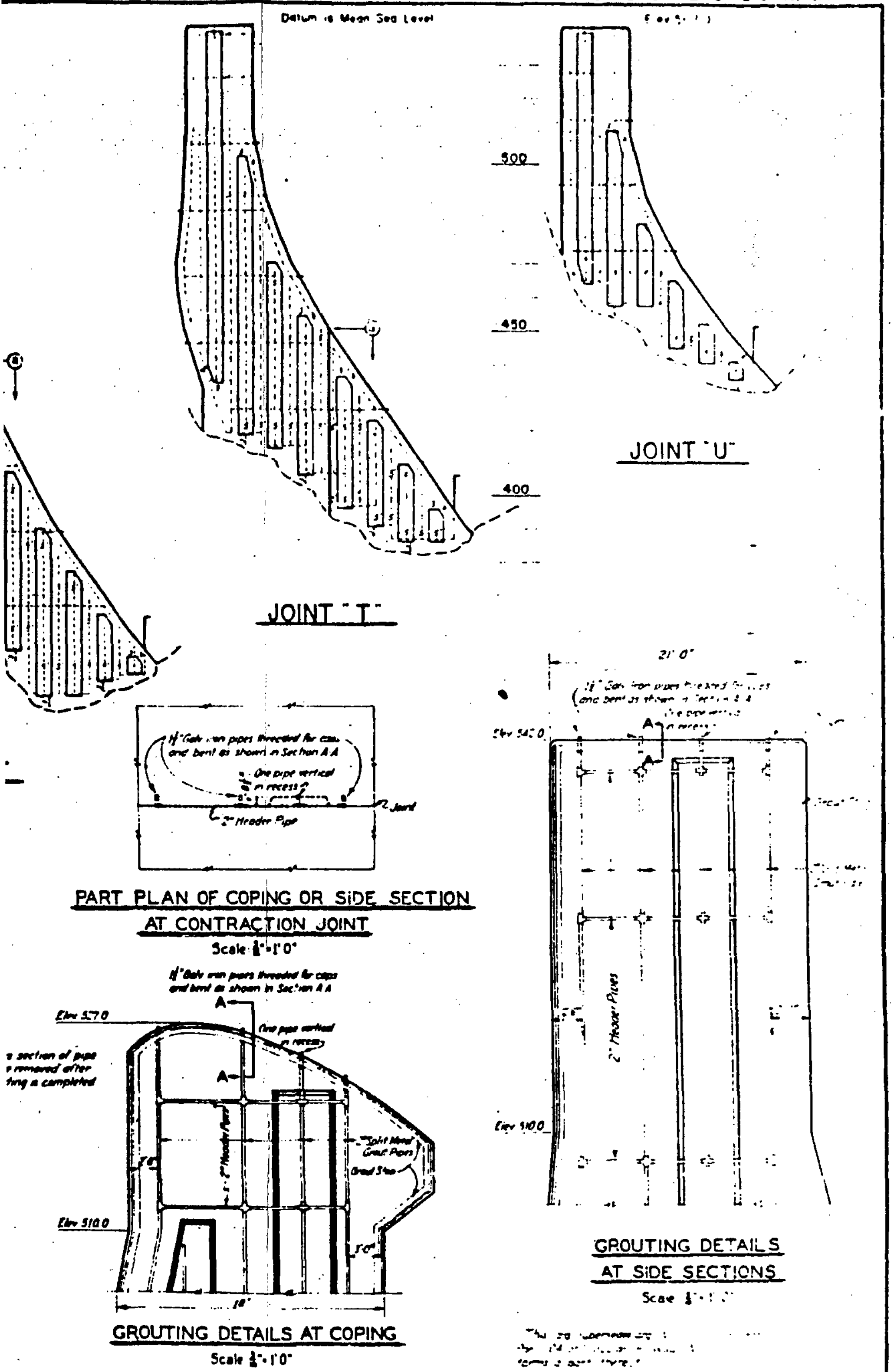
Submitted

Approved

71

2

THIS DRAWING SUPERSEDES DRG. No. 10, 8-9



TO ACCOMPANY CHANGE ORDER Nº 12
THE ARUNDEL CORPORATION & L E DIXON CO
CONTRACTORS, CONTRACT Nº 1105 ENG 24

REFERENCE DRAWINGS

Plan location and Cross Section — Drawing No 1, 8-9-246-2
Contraction Joints — Drawing Nos 7&8, 8-9-246-2
Aeration piers and coping details — Drawing No 1, 8-9-3?
Cable layout — Drawing No 1, 8-9-246-2

SACRAMENTO RIVER
AND TRIBUTARIES, CALIFORNIA
DEBRIS CONTROL
UPPER NARROWS DAM
YUBA RIVER

CROSS SECTIONS
CONTRACTION JOINTS

Scale 1" = 20'

IN 4 DRAWINGS SCALE AS SHOWN

U.S. Engineer Office, Sacramento, California

Submitted — Approved — Recommended

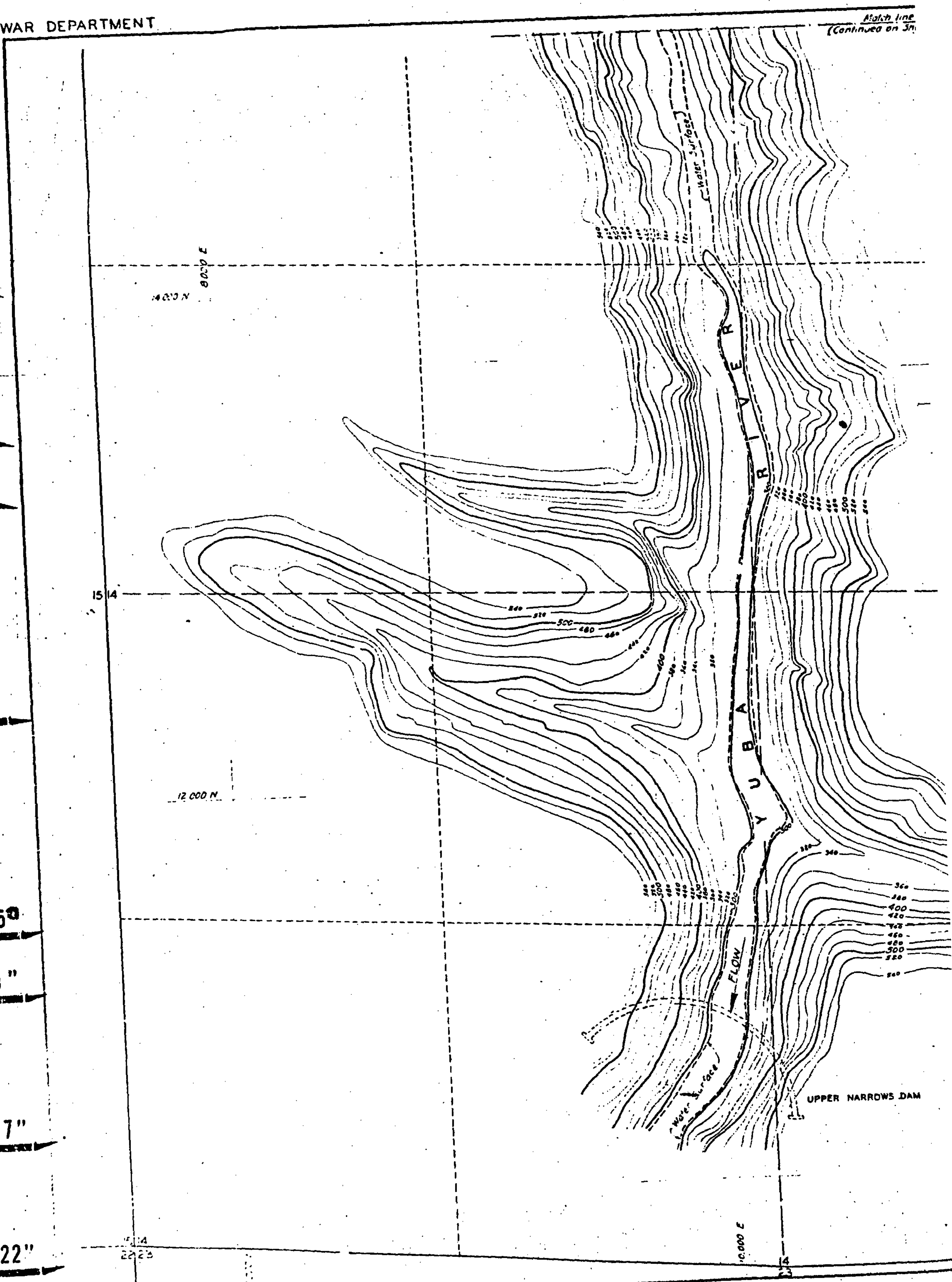
REVISION	DATE	DRG. NO.
Increased length of drain and grout...	June 1942	8-9-246-2
All keyways revised to vertical		8-9-246

71 — 8 9 29

3

THIS DRAWING SUPERSEDES DRG Nº 10, 8-9-246-2, DATED

WAR DEPARTMENT



22"

17"

11"

8.5"

8.5"

11"

17"

22"

1

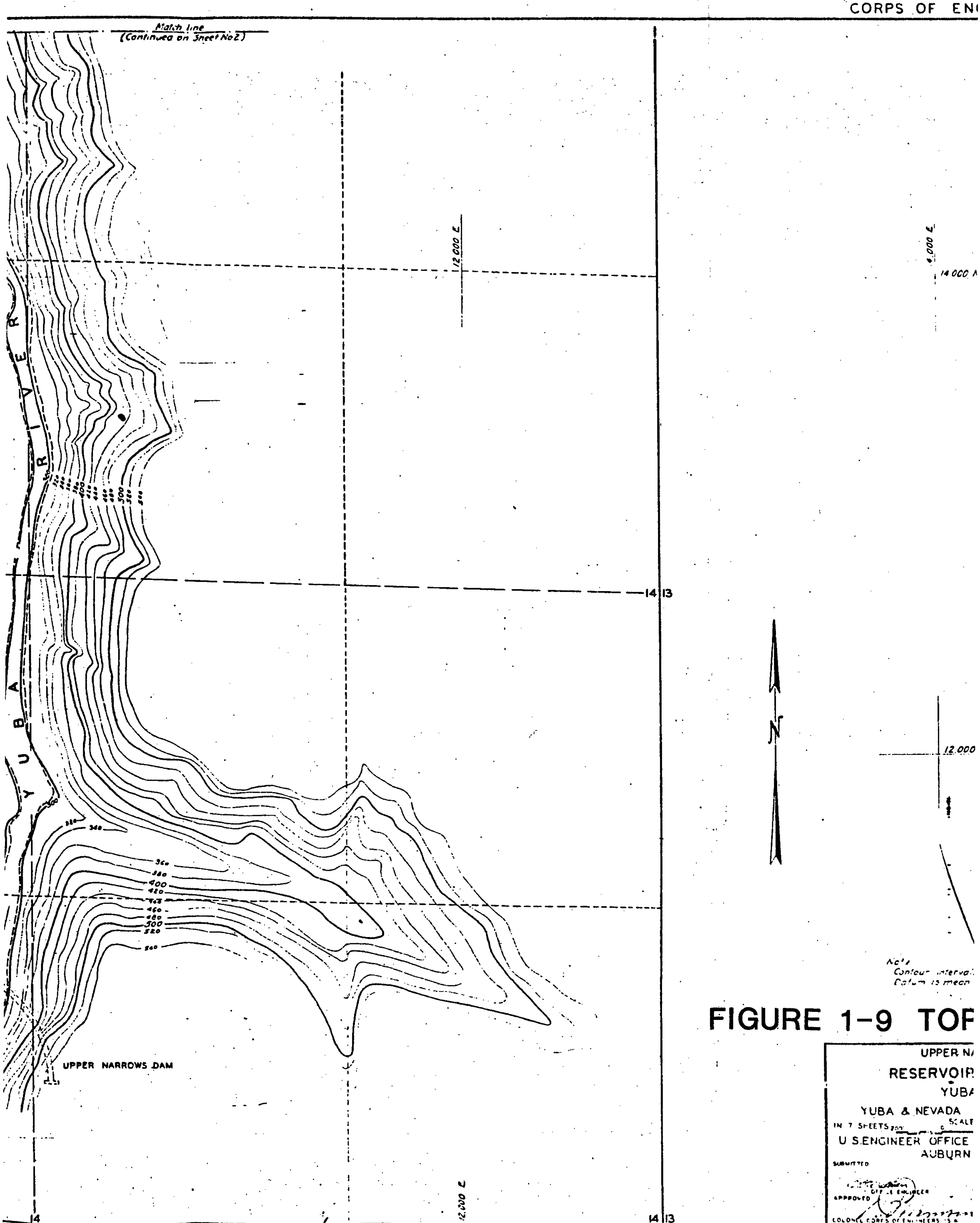


FIGURE 1-9 TOF

2

CORPS OF ENGINEERS, U.S. ARMY

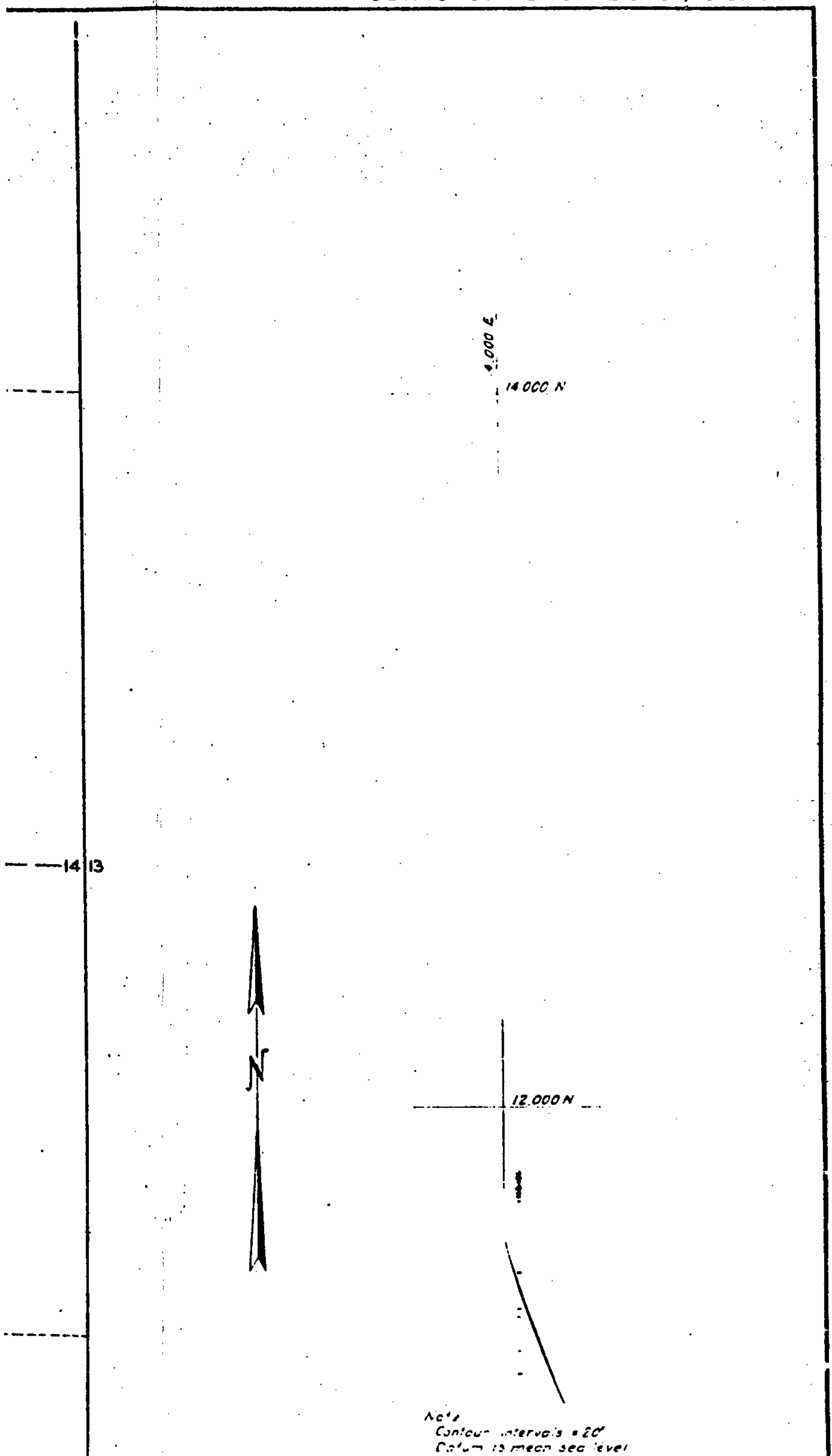


FIGURE 1-9 TOPOGRAPHY

UPPER NARROWS DAM

RESERVOIR TOPOGRAPHY

YUBA RIVER

YUBA & NEVADA COUNTIES, CALIFORNIA

IN 7 SHEETS SCALE 1 IN = 200 FT SHEET NO

U.S. ENGINEER OFFICE, SACRAMENTO DISTRICT

AUBURN AREA OFFICE

SUBMITTED APPROVAL RECOMMENDED

OFFICE ENGINEER CONSTRUCTION ENGINEER

APPROVED

COLONEL CORPS OF ENGINEERS U.S.A.

DOC CAT	FILE	DIV	SHEET
	8	9	300

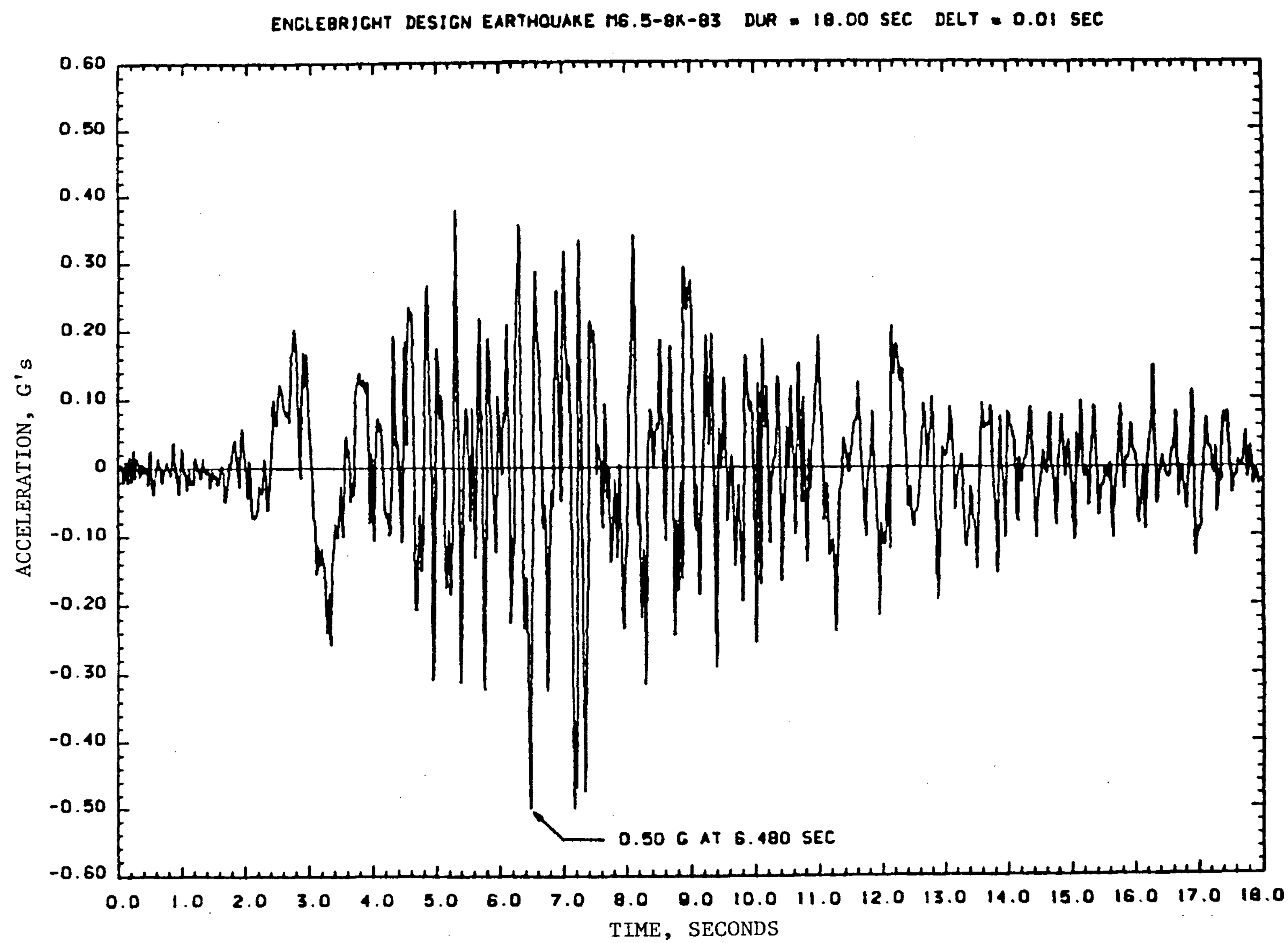


FIGURE 3-1
M6.5-8K-83 DELTA T=0.01 SEC

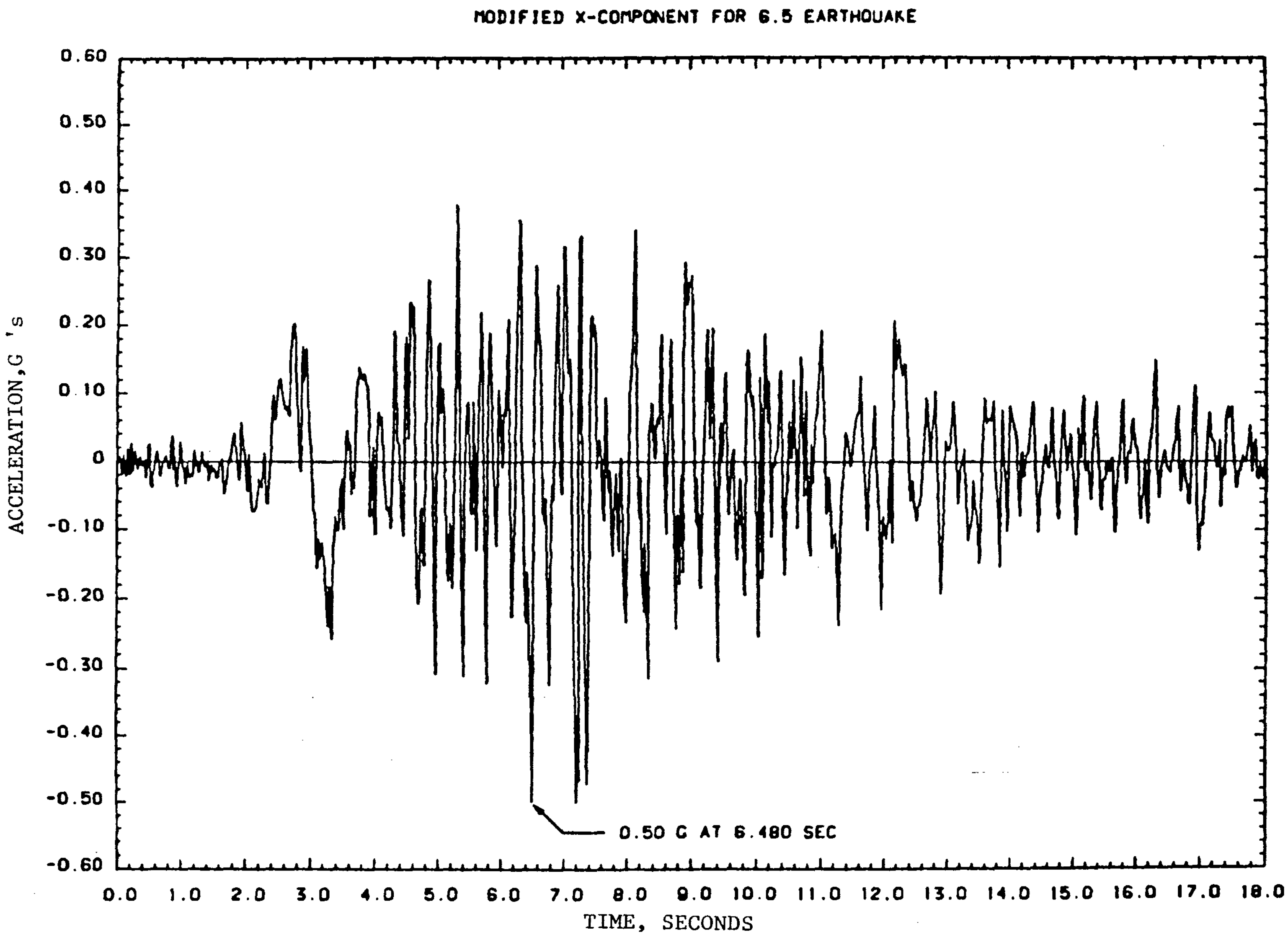


FIGURE 3-2
M6.5-8K-83 DELTA T=0.02 SEC

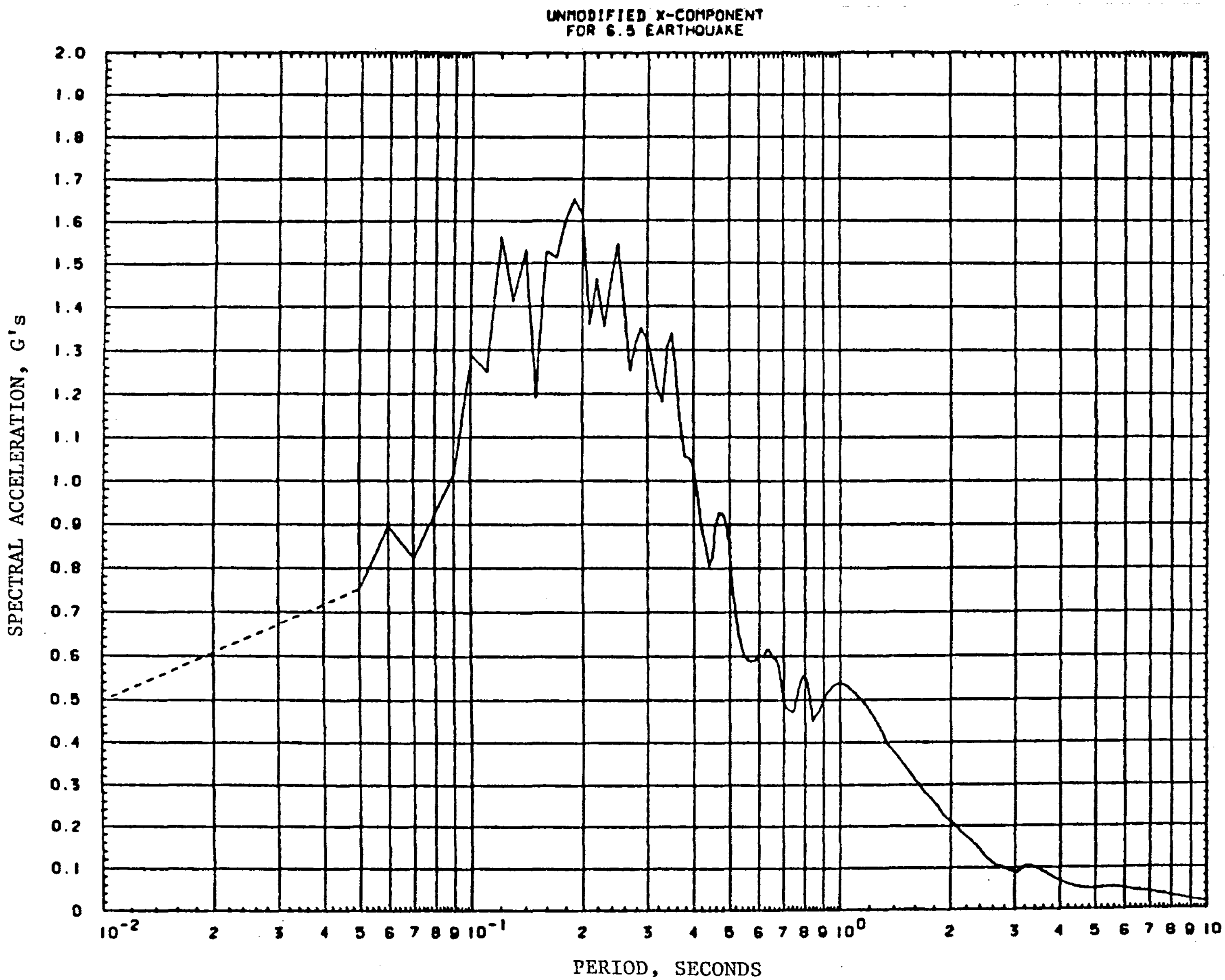


FIGURE 3-3
M6.5-8K-83 DELTA T=0.01 SEC.

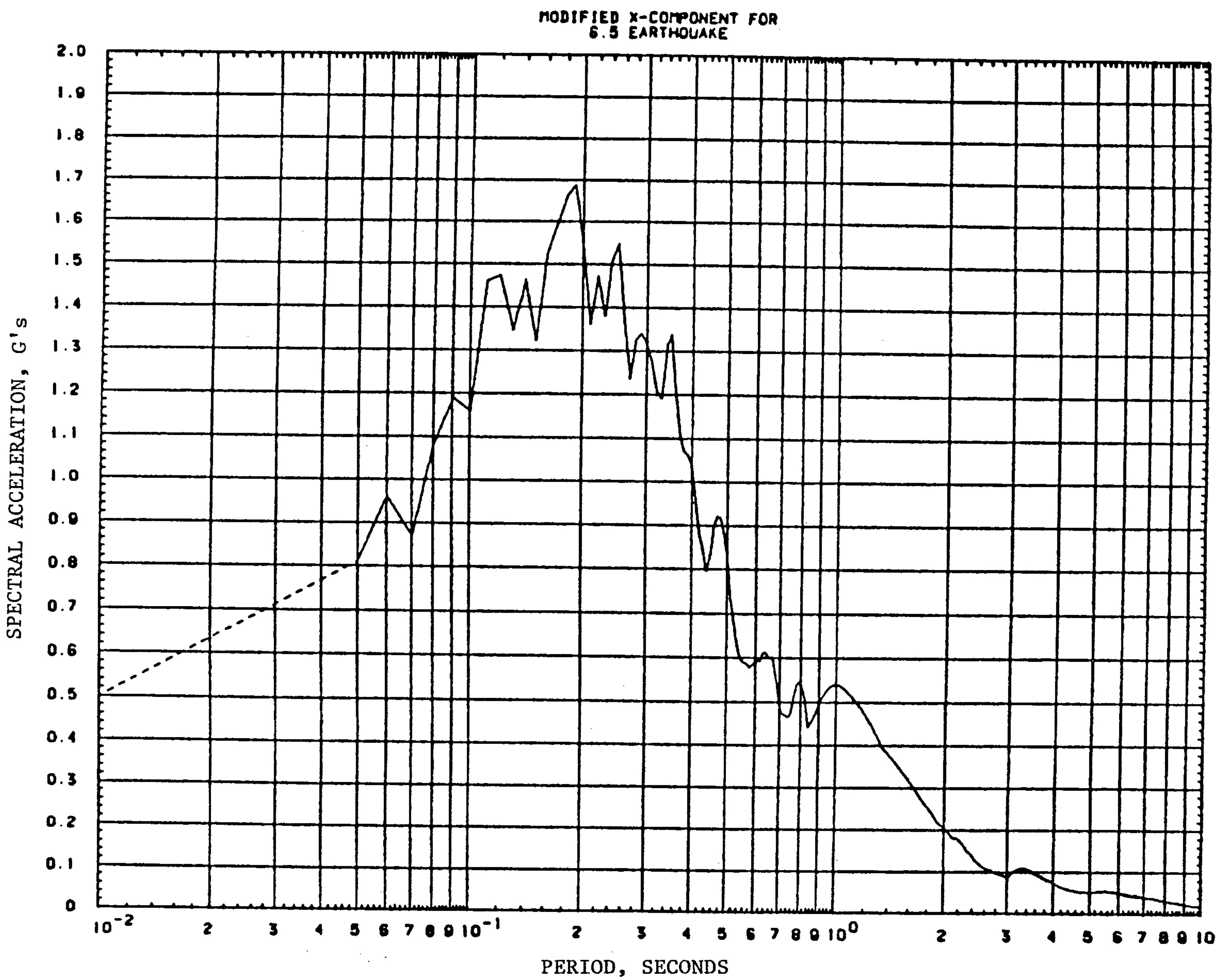
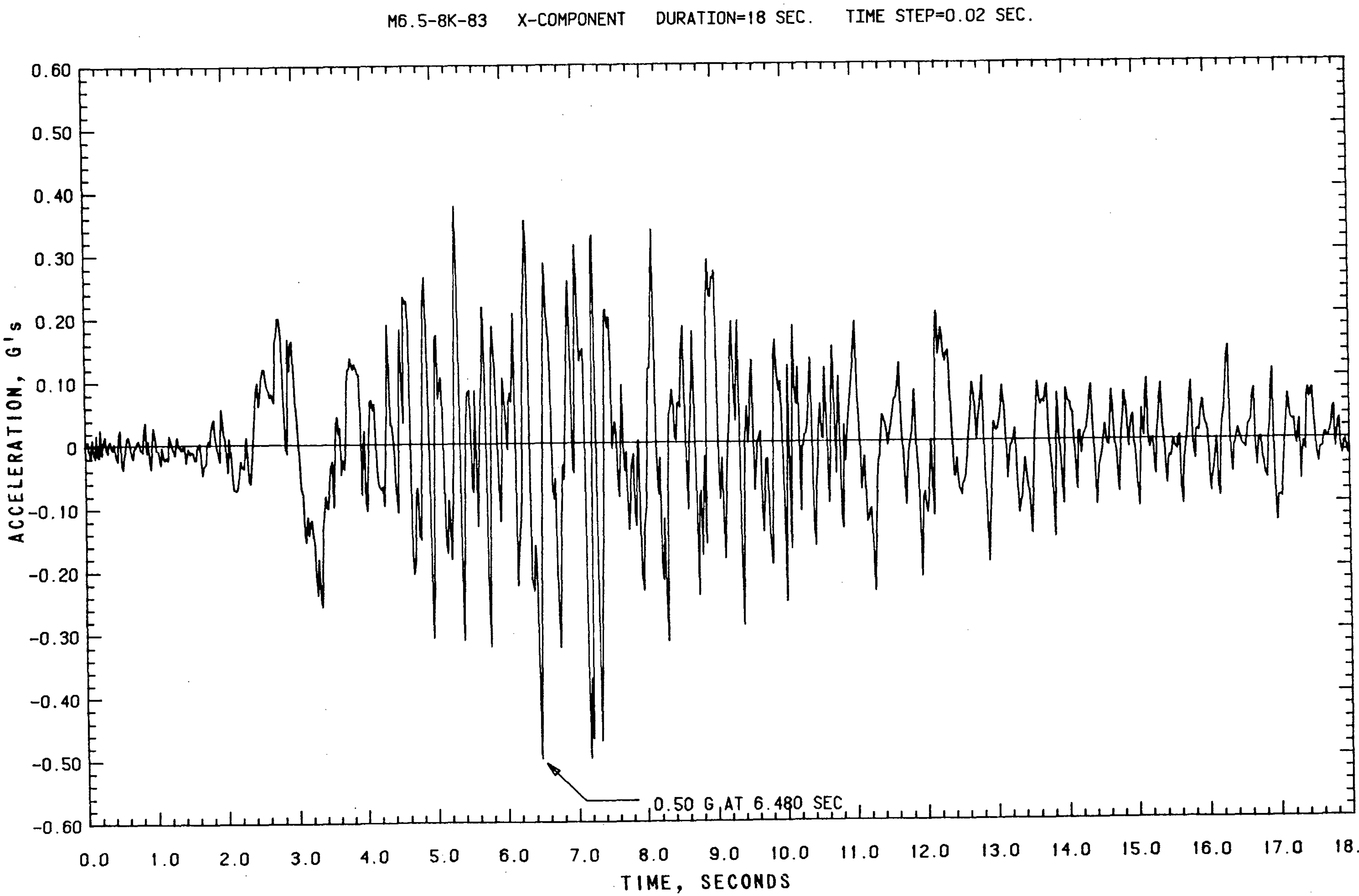


FIGURE 3-4
M6.5-8K-83 DELTA T=0.02 SEC



M6.5-8K-83 X-COMPONENT DURATION=18 SEC. TIME STEP=0.02 SEC.
ACCELERATION, G's
0.60
0.50
0.40
0.30
0.20
0.10
0
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
0.50 G AT 6.480 SEC
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
17.0
18.0
TIME, SECONDS

FIGURE 3-5

M6.5-8K-83   Y-COMPONENT   DURATION=12 SEC.   TIME STEP=0.02 SEC.

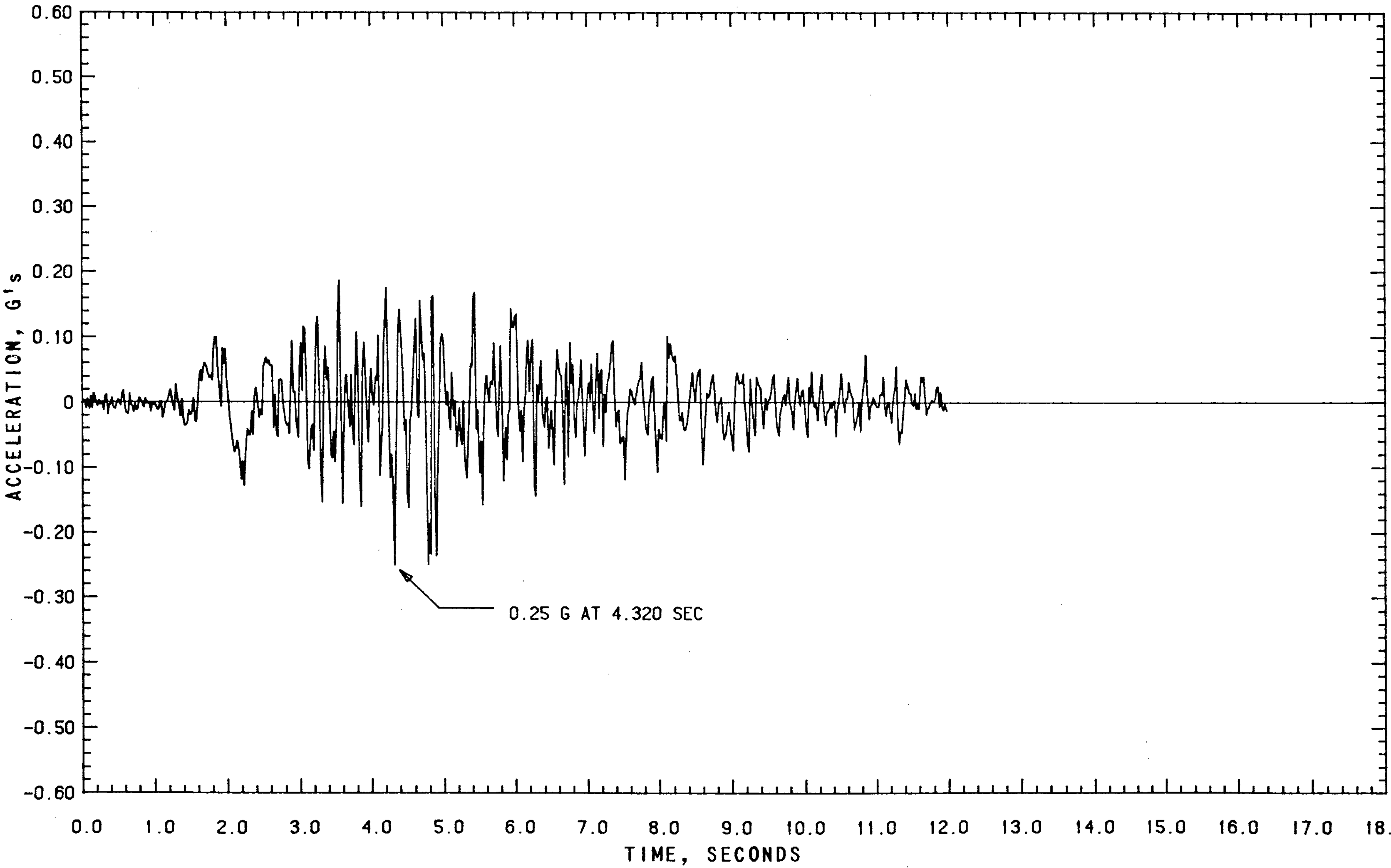


FIGURE 3-6

M6.5-8K-83 Z-COMPONENT DURATION=18.45 SEC. TIME STEP=0.02 SEC.

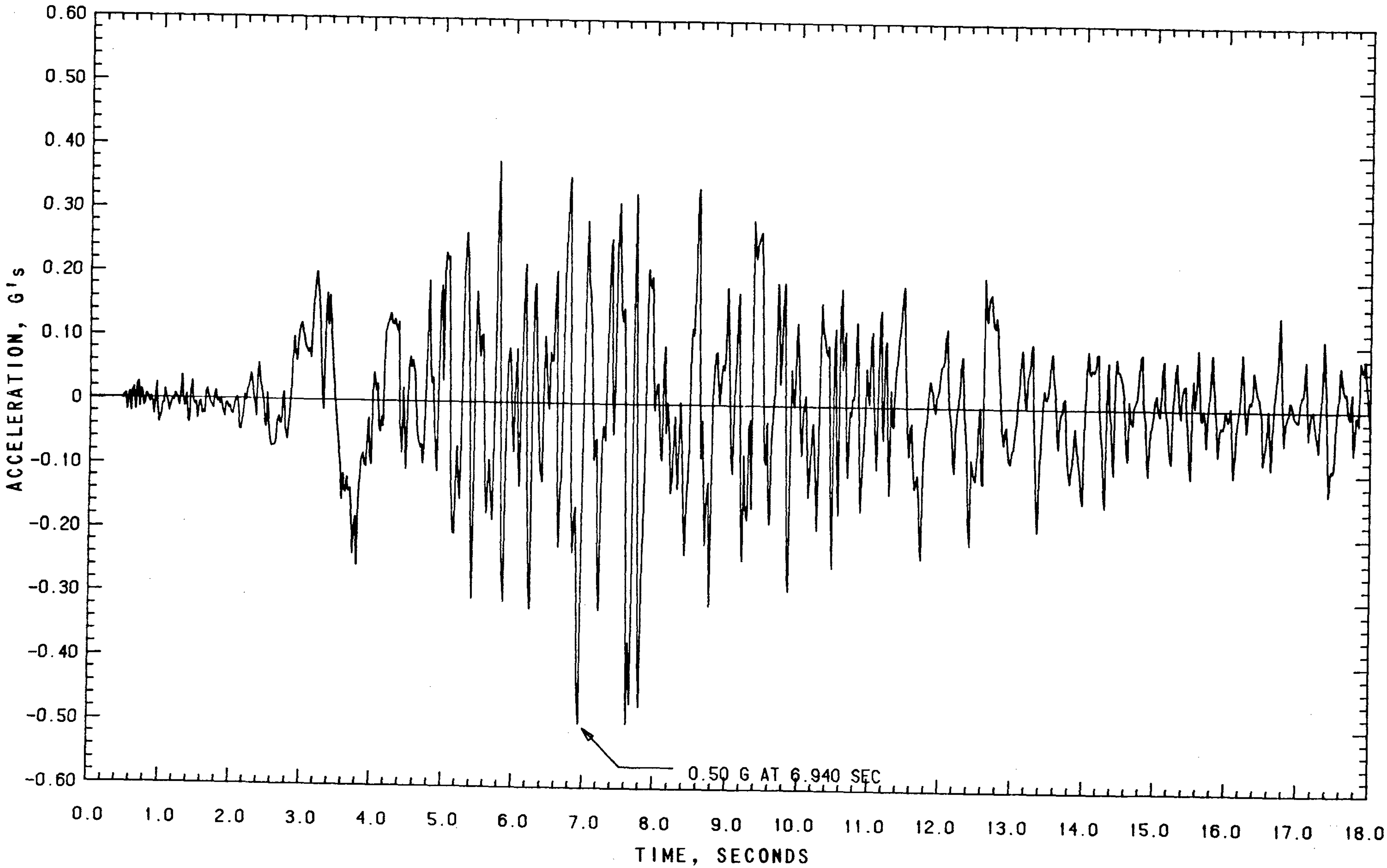


FIGURE 3-7

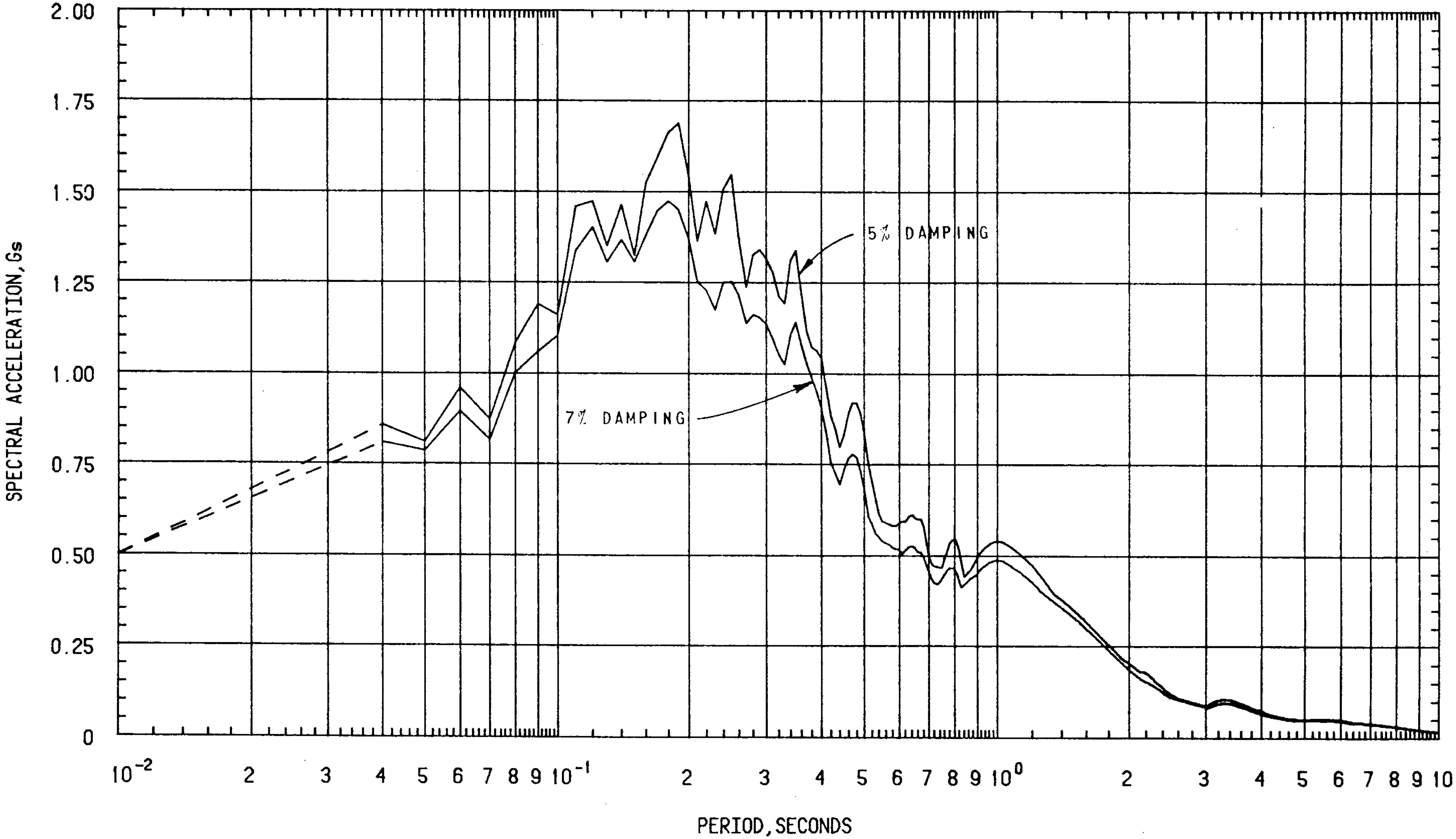


FIGURE 3-8

M6.5-8k-83 X-COMPONENT

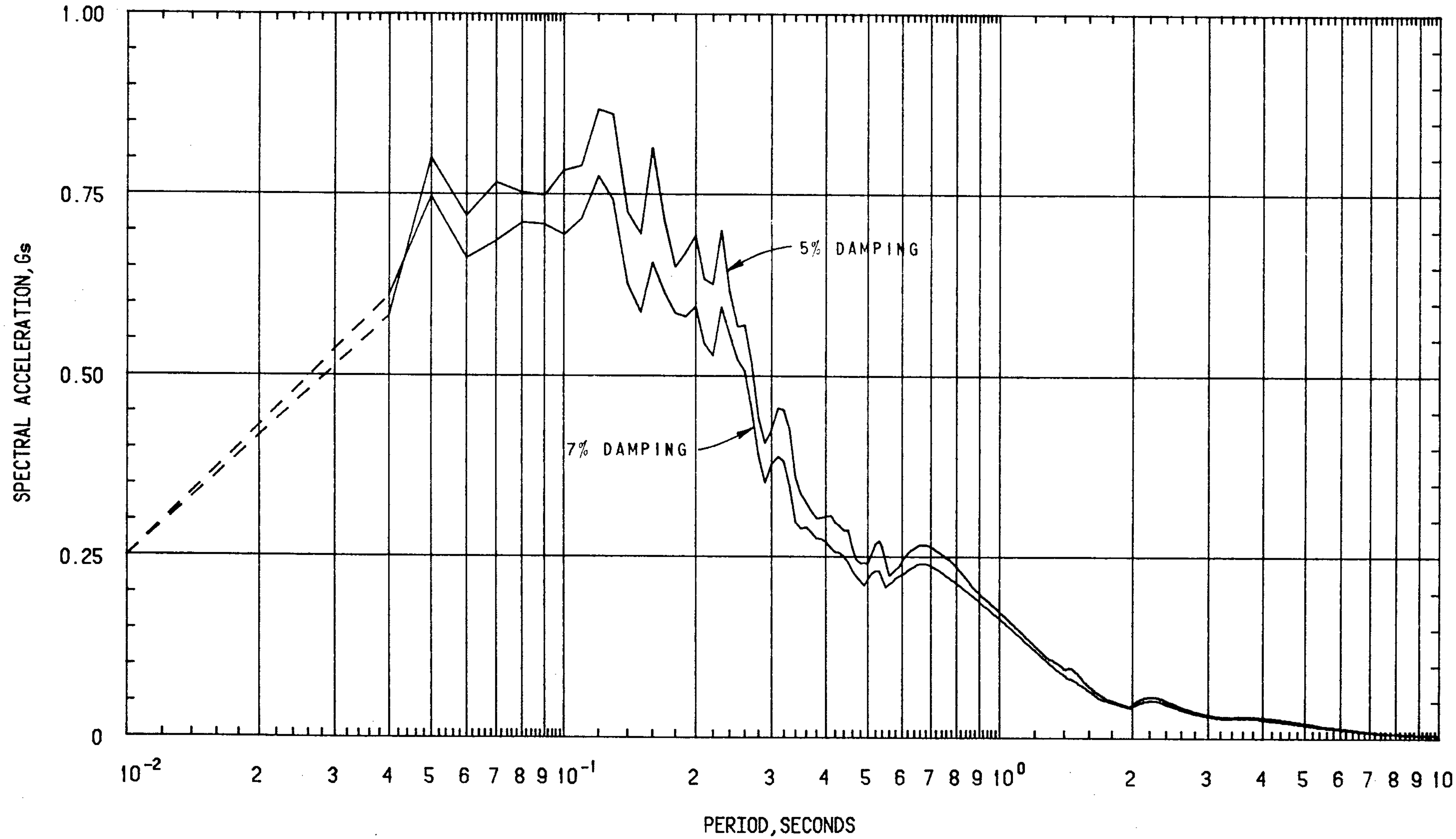


FIGURE 3-9

M6.5-8k-83 Y-COMPONENT

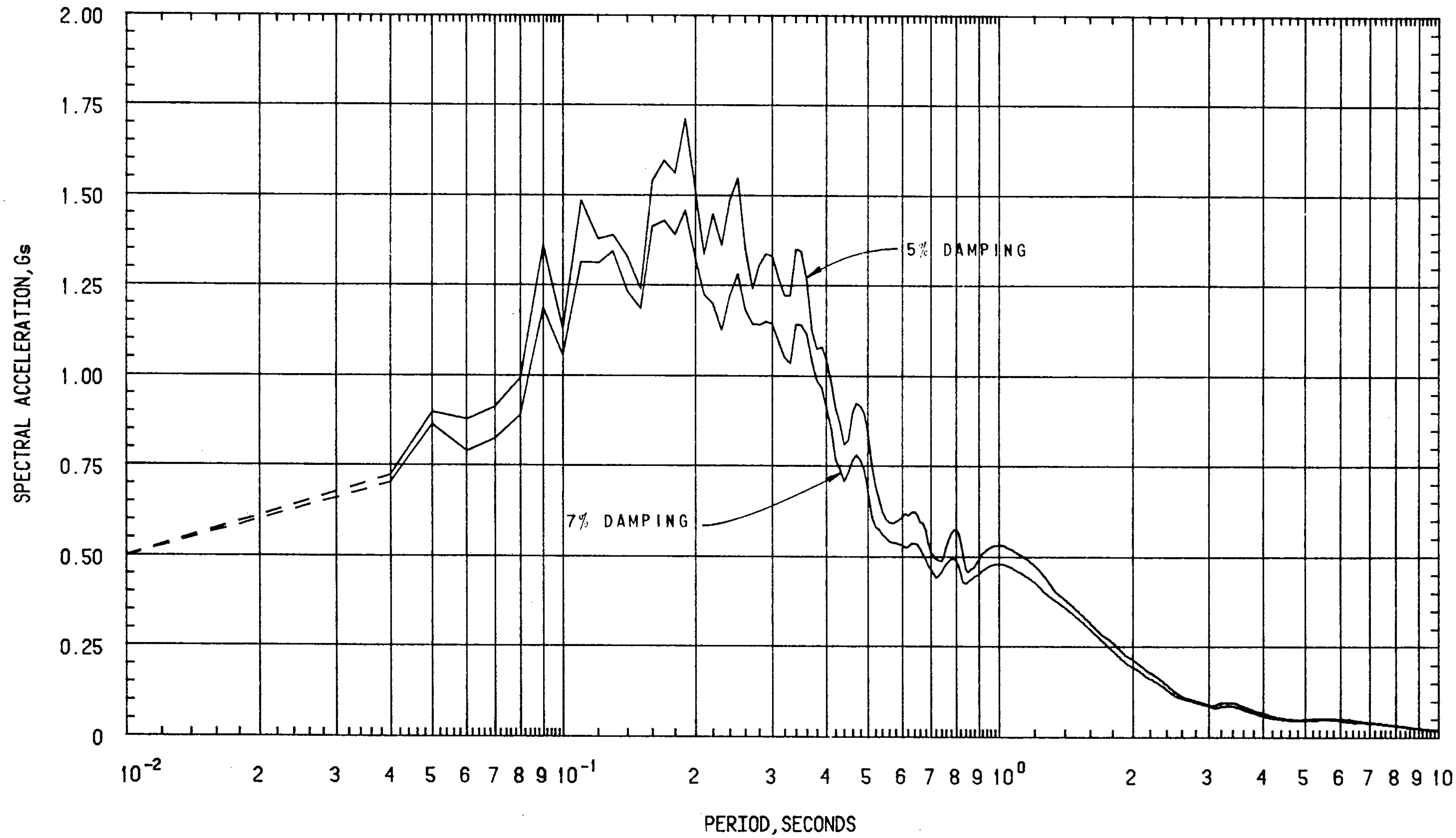


FIGURE 3-10

M6.5-8k-83 Z-COMPONENT

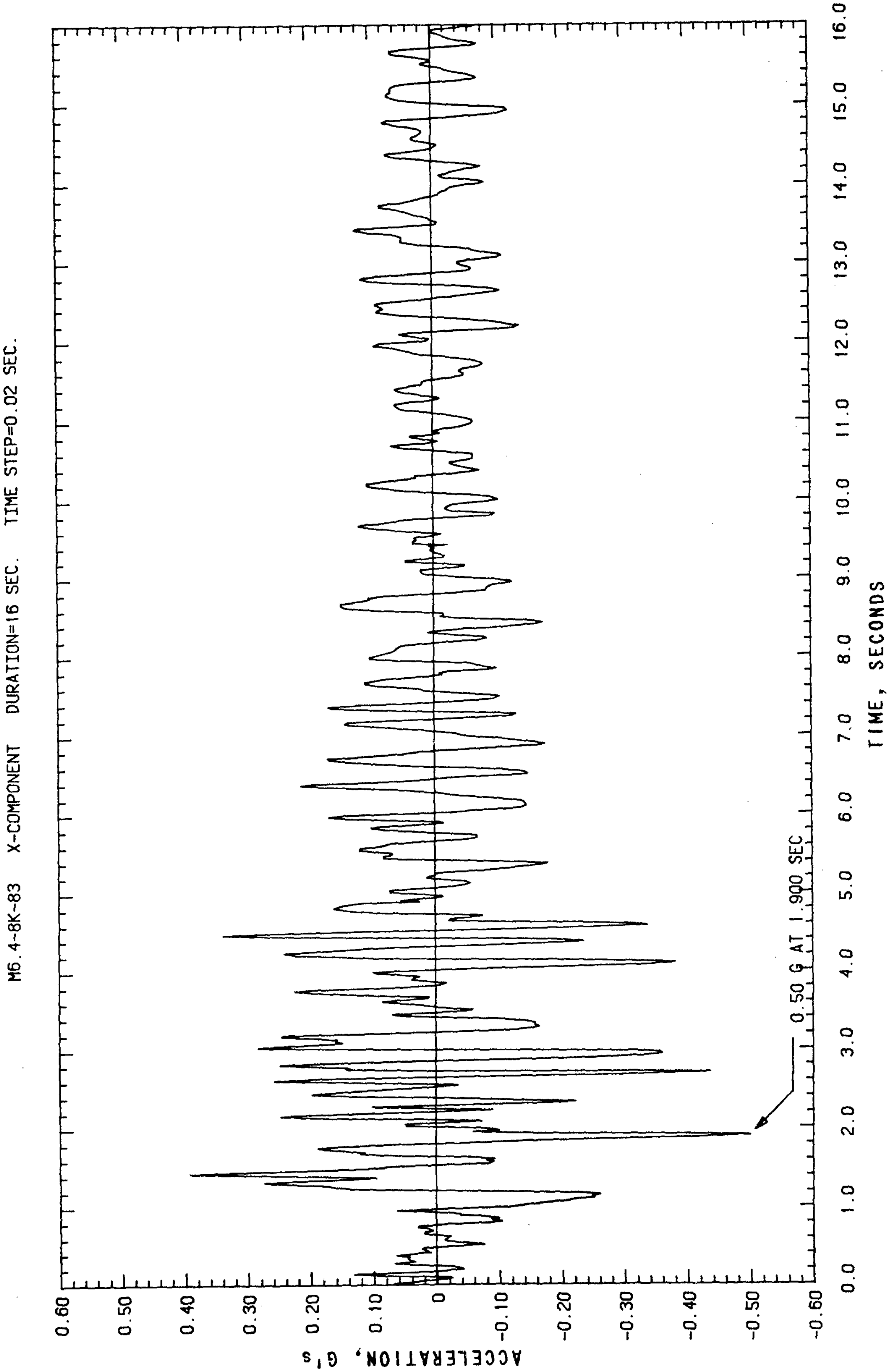

M6.4-8K-83 X-COMPONENT DURATION=16 SEC. TIME STEP=0.02 SEC.
0.50 G AT 1.900 SEC
ACCELERATION, G's
TIME, SECONDS
0.60
0.50
0.40
0.30
0.20
0.10
0
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0

FIGURE 3-11

M6.4-8K-83 Y-COMPONENT DURATION=10.67 SEC TIME STEP=0.02 SEC

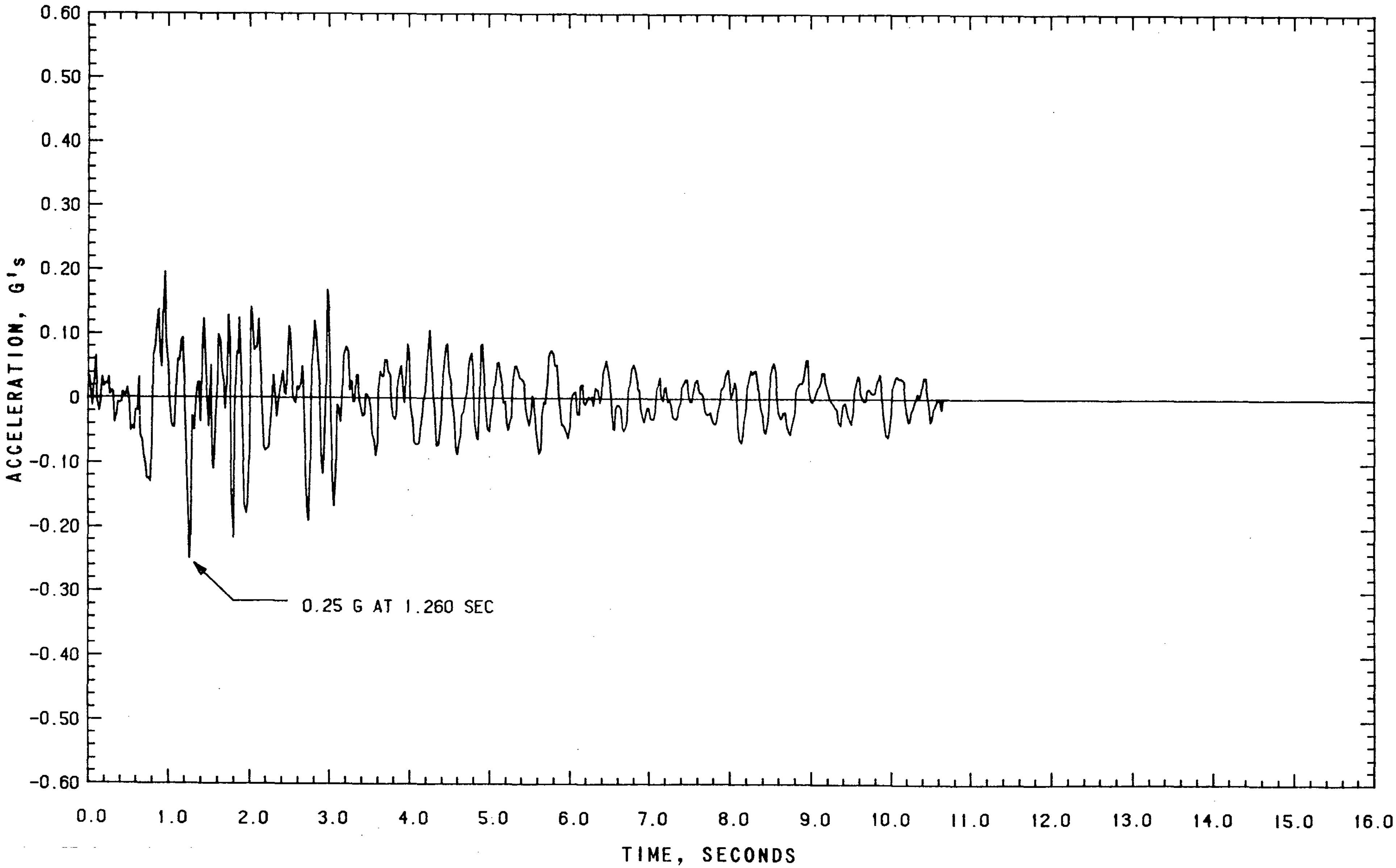
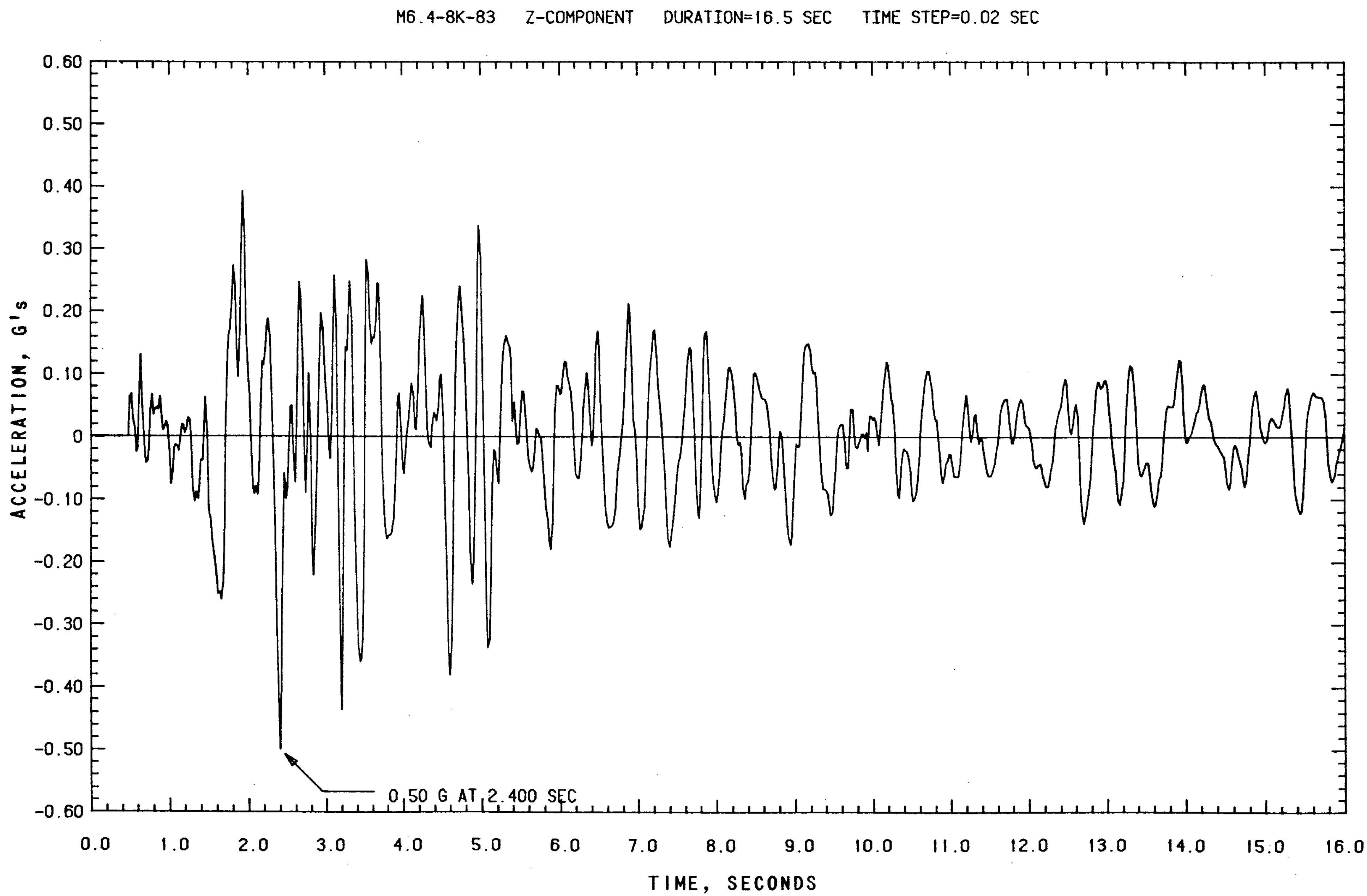


FIGURE 3-12


M6.4-8K-83 Z-COMPONENT DURATION=16.5 SEC TIME STEP=0.02 SEC
ACCELERATION, G's
0.60
0.50
0.40
0.30
0.20
0.10
0
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
0.50 G AT 2.400 SEC
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
TIME, SECONDS
FIGURE 3-13

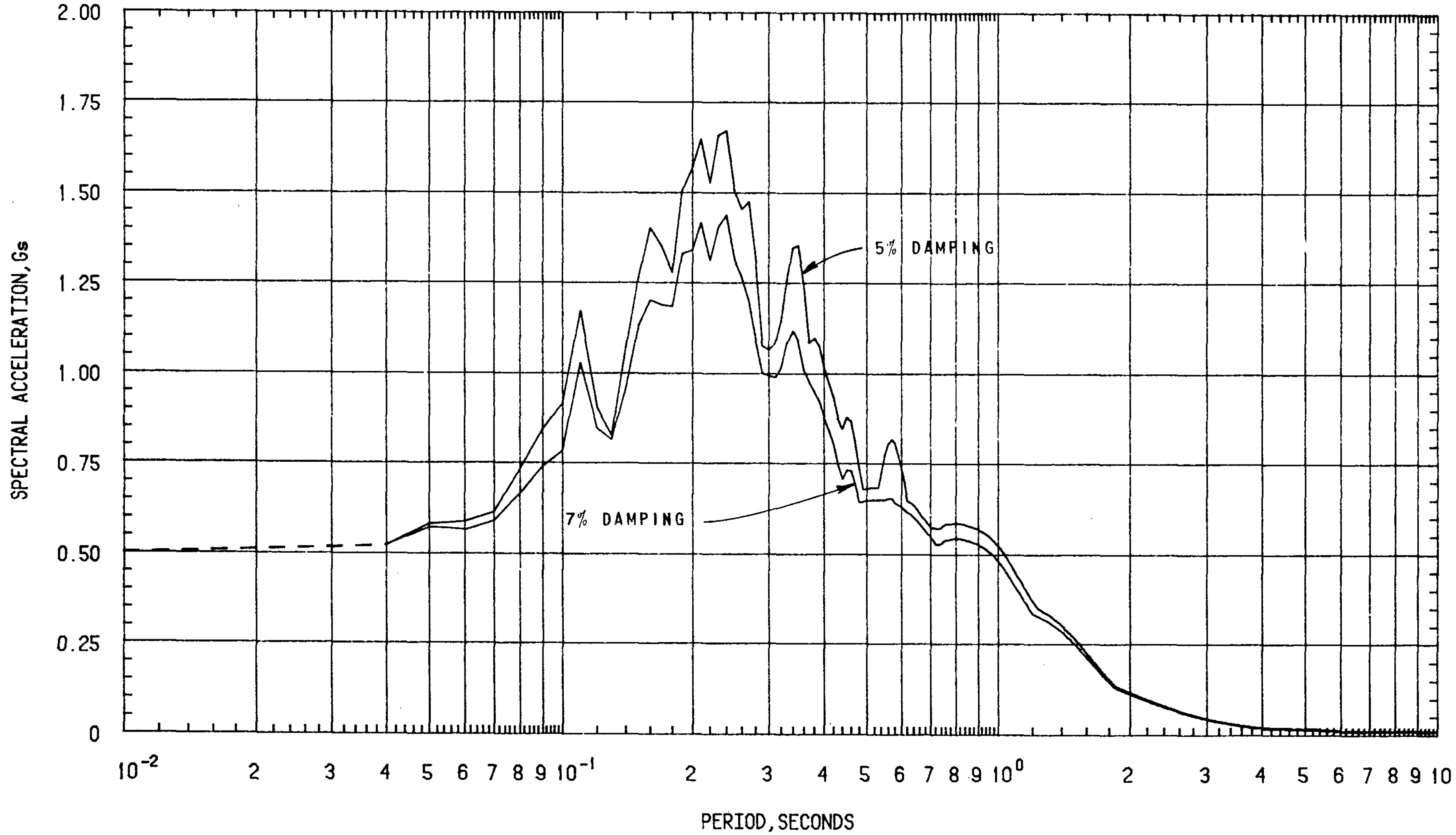


FIGURE 3-14

M6.4-8k-83 X-COMPONENT

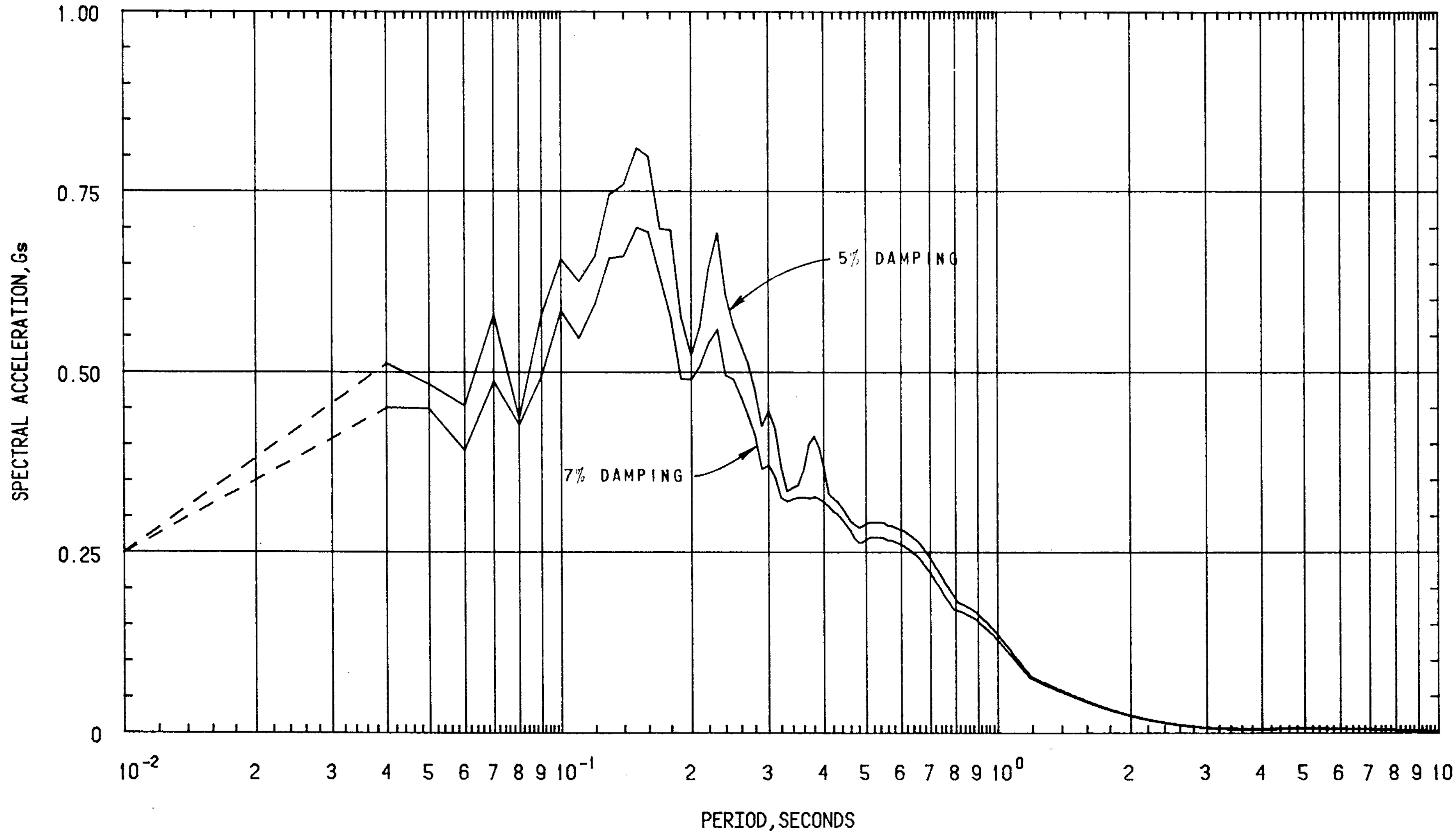


FIGURE 3-15

M6.4-8k-83 Y-COMPONENT

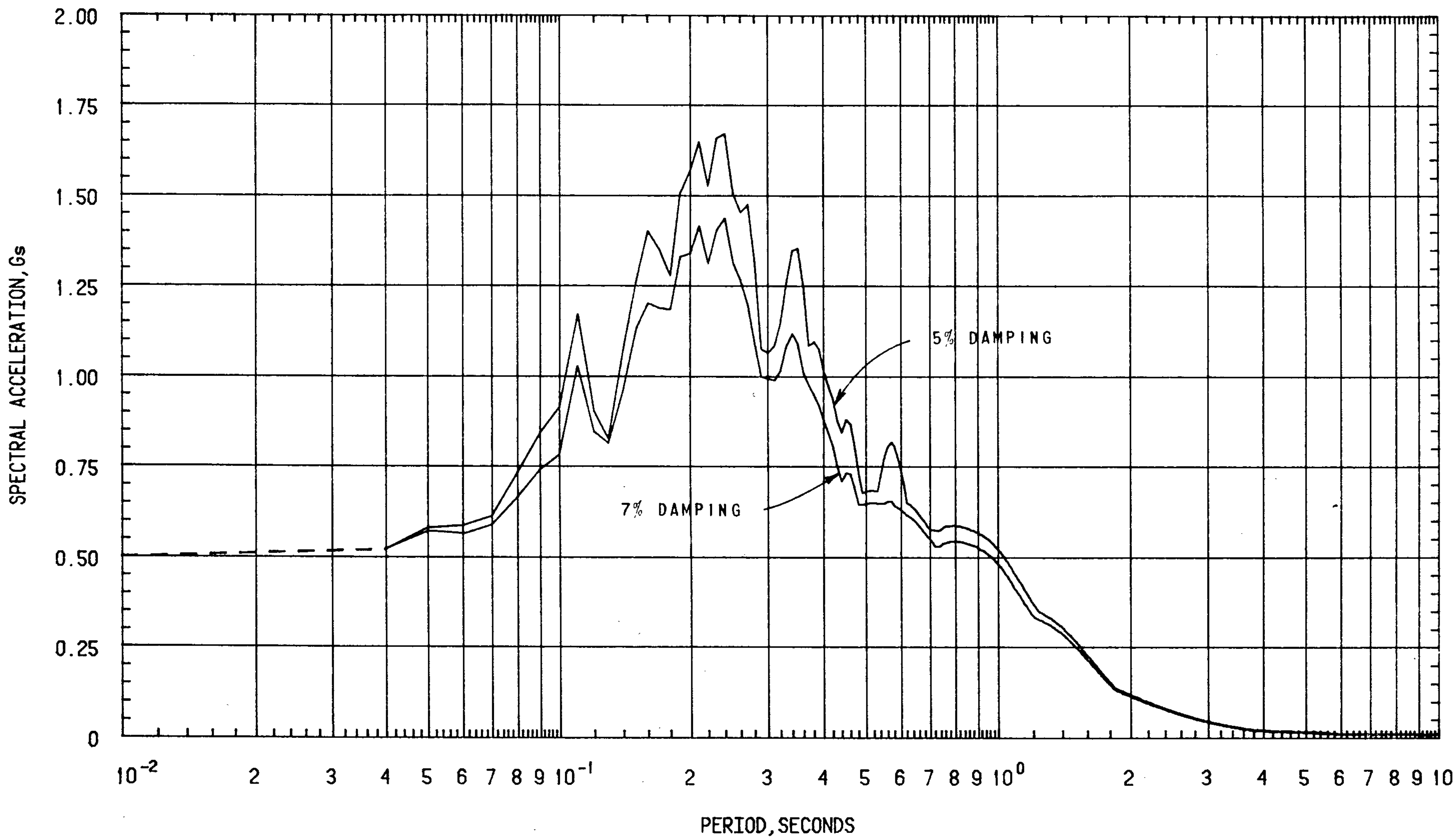
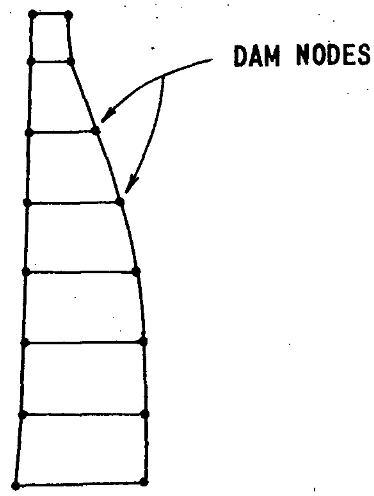
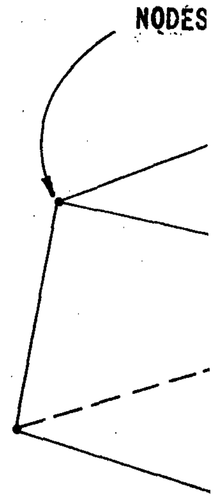


FIGURE 3-16

M6.4-8k-83 Z-COMPONENT



CROWN CANTILEVER SECTION



TYPICAL 8 -

NOTE:
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SIX QU

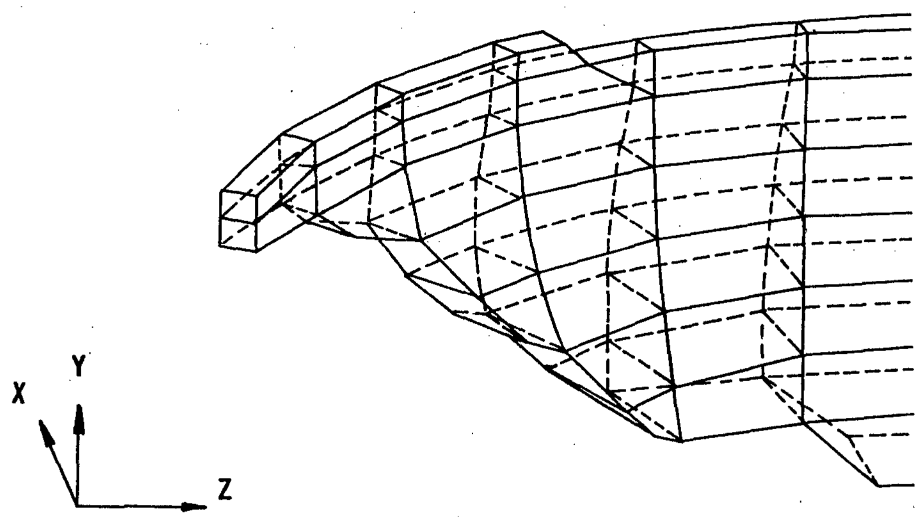
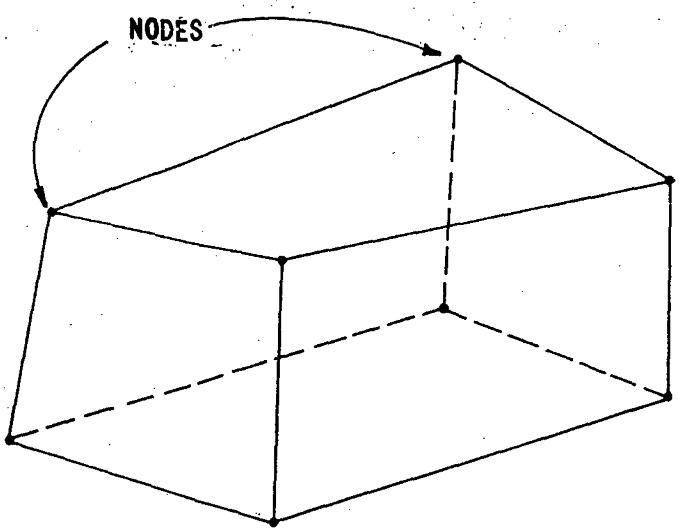
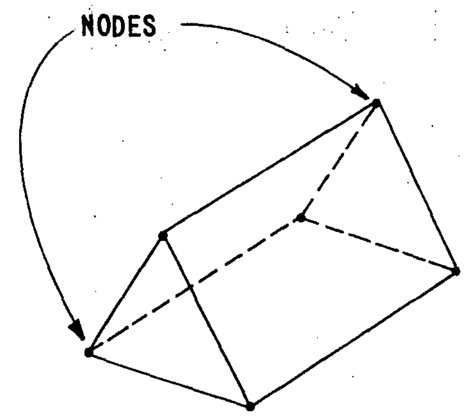


FIGURE 6-1 1-LAYER-8-NODE S



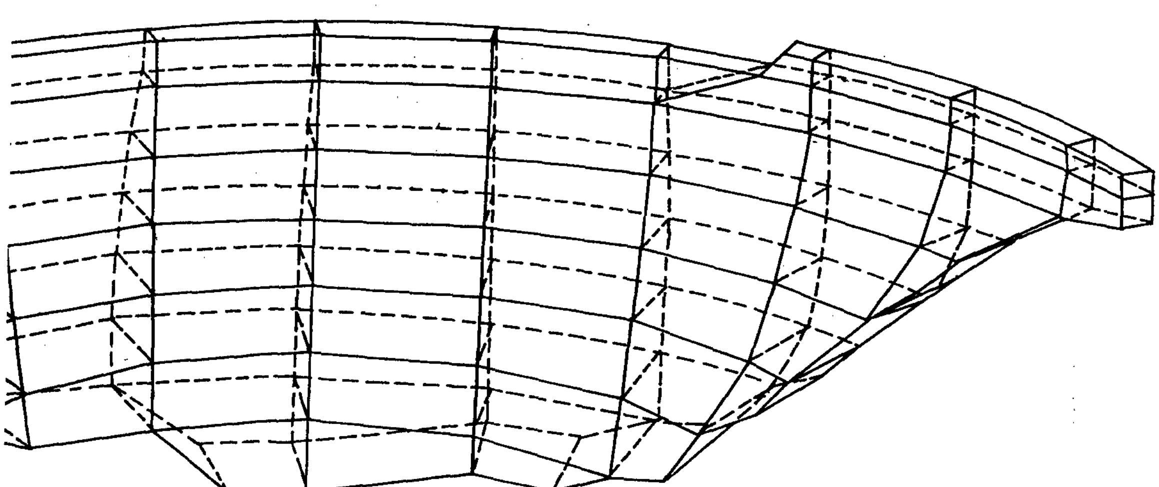
TYPICAL 8 - NODE SOLID DAM ELEMENT

NOTE:
THIS TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL FACES.



TYPICAL 6 - NODE PRISMATIC SOLID DAM ELEMENT

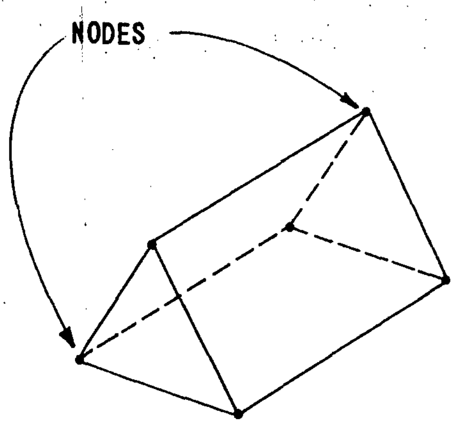
NOTE:
THIS TYPE OF SOLID ELEMENT HAS TWO TRIANGULAR FACES AND THREE QUADRILATERAL FACES.



TOTAL NUMBER OF DAM ELEMENTS = 65

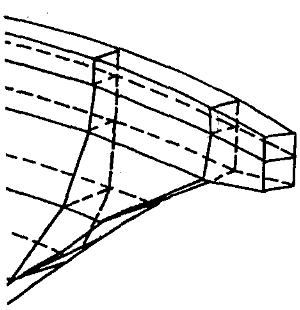
TOTAL NUMBER OF DAM NODES = 174

I 1-LAYER-8-NODE SOLID FINITE ELEMENT DAM MODEL (1L8ND)



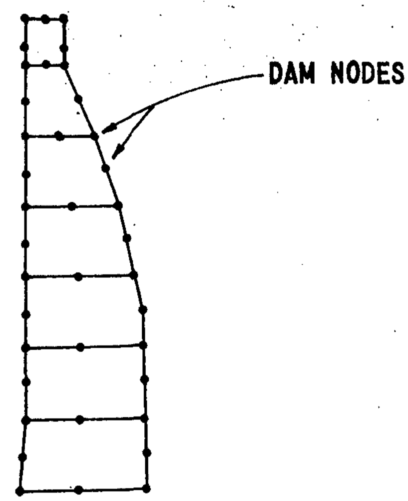
TYPICAL 6 - NODE PRISMATIC SOLID DAM ELEMENT

NOTE:
THIS TYPE OF SOLID ELEMENT HAS TWO TRIANGULAR FACES AND THREE QUADRILATERAL FACES.



TOTAL NUMBER OF DAM ELEMENTS = 65

TOTAL NUMBER OF DAM NODES = 174



CROWN CANTILEVER SECTION

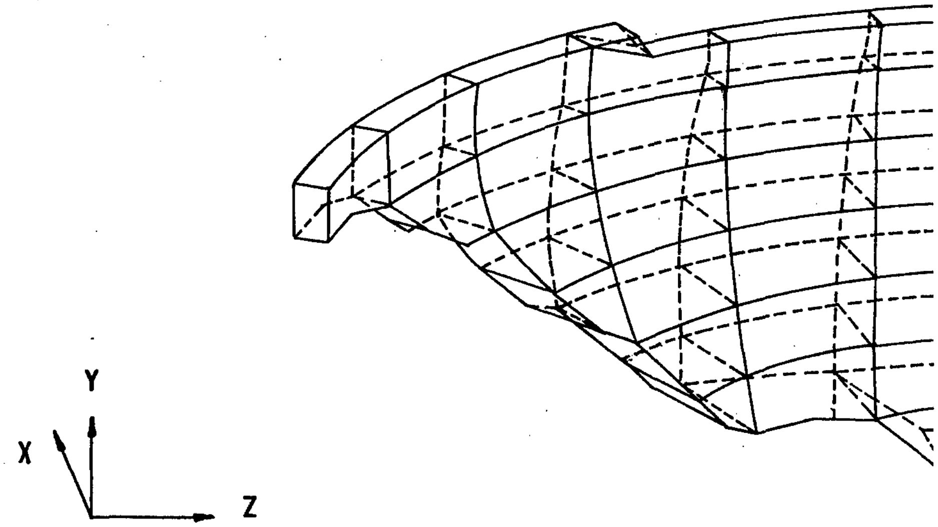
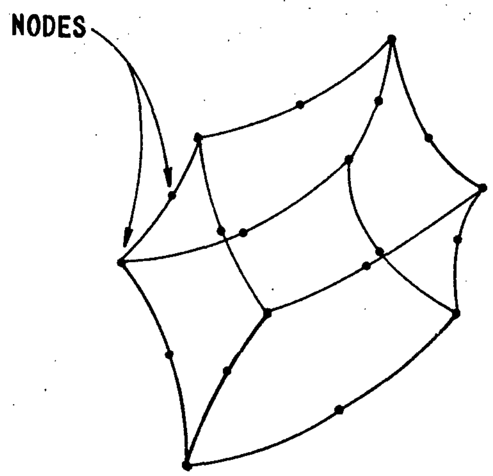
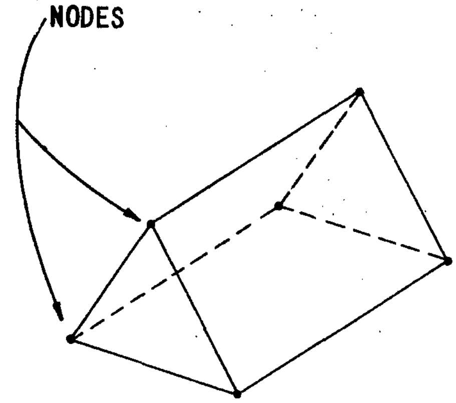


FIGURE 6-2 I-LAYER-2(



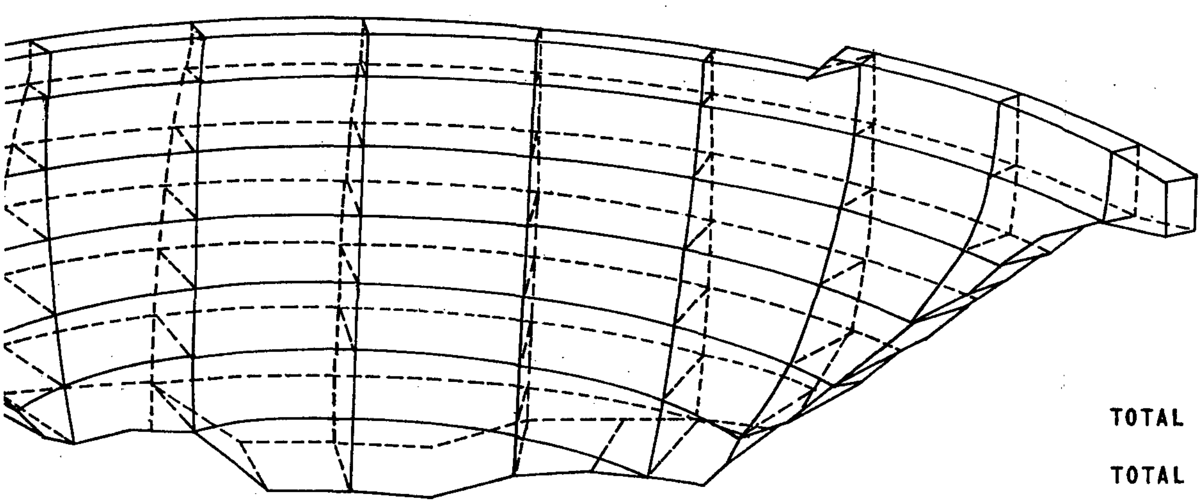
TYPICAL 20 - NODE SOLID DAM ELEMENT

NOTE:
THIS TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL FACES. THE EDGES OF THE ELEMENT MAY BE CURVED TO A PARABOLIC SHAPE SINCE THERE ARE THREE NODES ALONG EACH EDGE.



TYPICAL 6 - NODE PRISMATIC SOLID DAI

NOTE:
THIS TYPE OF SOLID ELEMENT HAS TRIANGULAR FACES AND THREE QUADI LATERAL FACES.

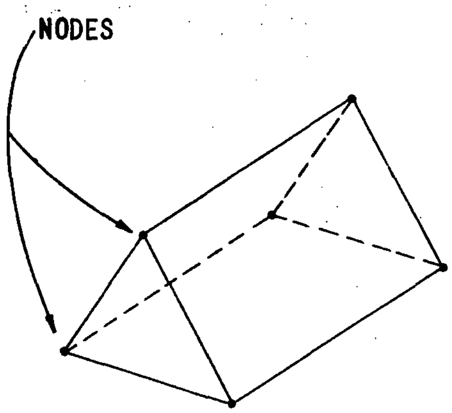


TOTAL NUMBER OF DAM ELEMENTS =

TOTAL NUMBER OF DAM NODES = 4

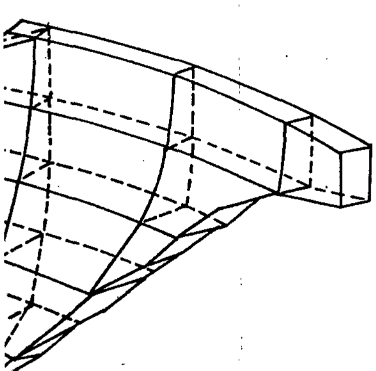
RE 6-2 I-LAYER-20-NODE SOLID FINITE ELEMENT DAM MODEL (IL20ND)

2



TYPICAL 6 - NODE PRISMATIC SOLID DAM ELEMENT

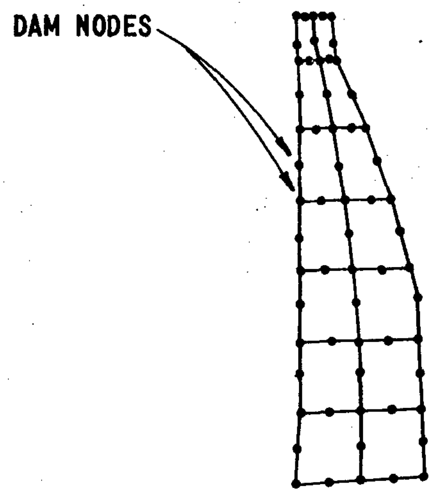
NOTE:
THIS TYPE OF SOLID ELEMENT HAS TWO TRIANGULAR FACES AND THREE QUADRILATERAL FACES.



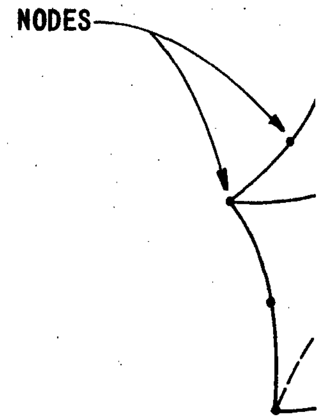
TOTAL NUMBER OF DAM ELEMENTS = 57

TOTAL NUMBER OF DAM NODES = 487

(IL20ND)



CROWN CANTILEVER SECTION



TYPICAL

NOTE:

THIS T
SIX QU
EDGES
CURVED
SINCE
ALONG

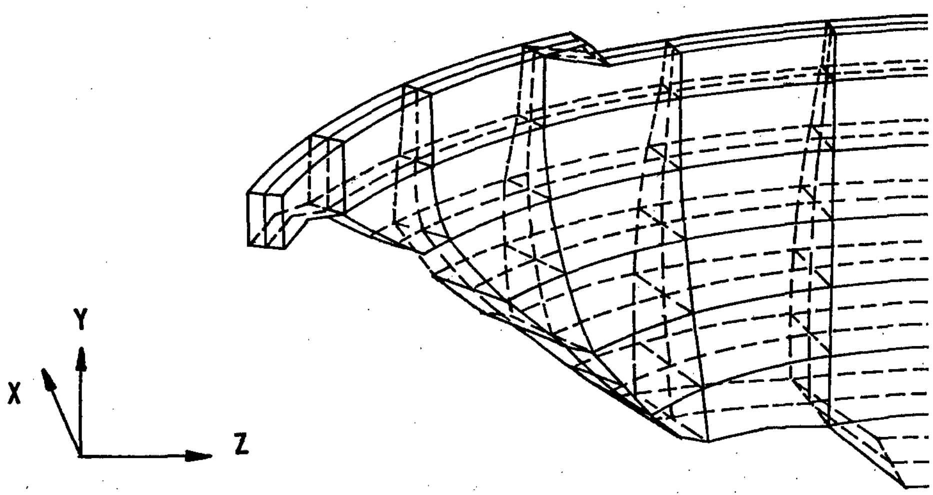
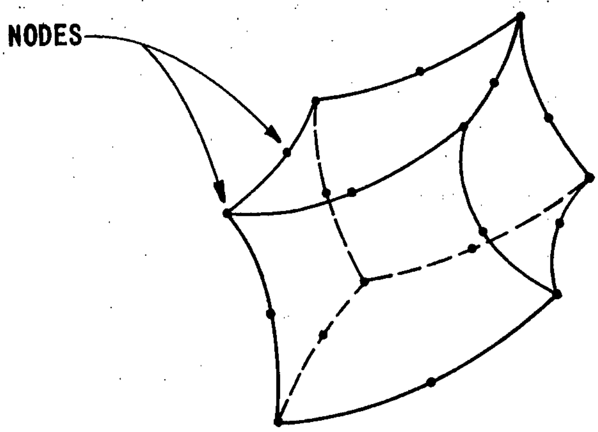


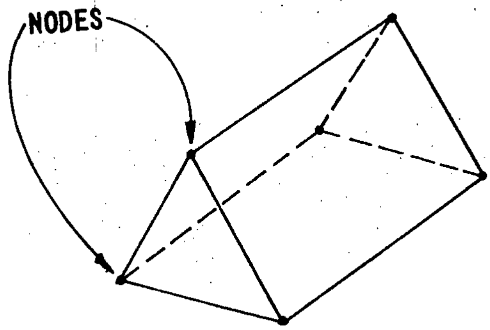
FIGURE 6-3 2-LAYER-20-



TYPICAL 20-NODE SOLID DAM ELEMENT

NOTE:

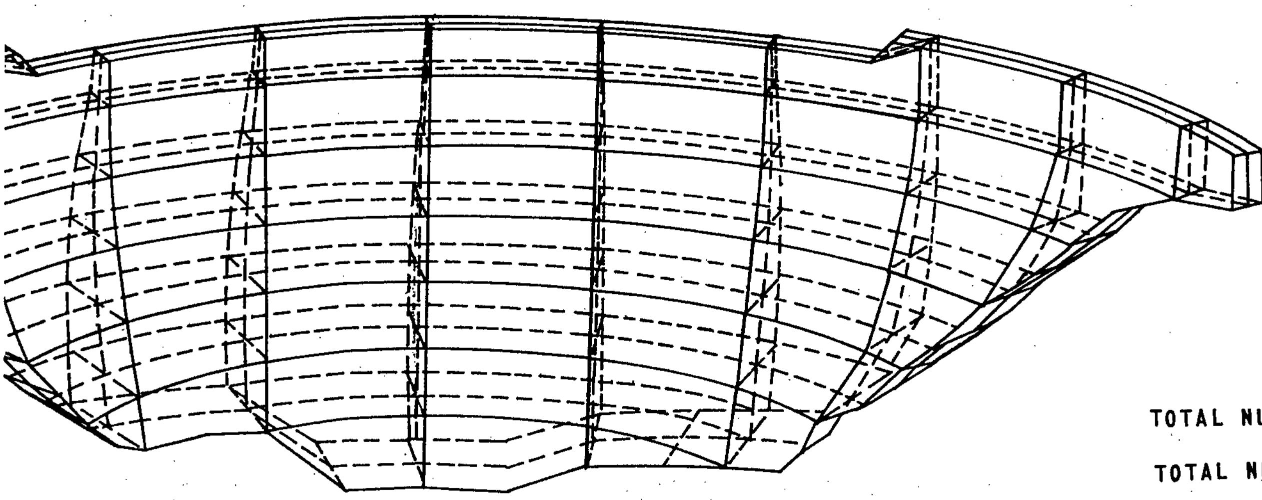
THIS TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL FACES. THE EDGES OF THE ELEMENT MAY BE CURVED TO A PARABOLIC SHAPE SINCE THERE ARE THREE NODES ALONG EACH EDGE.



TYPICAL 6-NODE PRISMATIC SOLID DAM ELEI

NOTE:

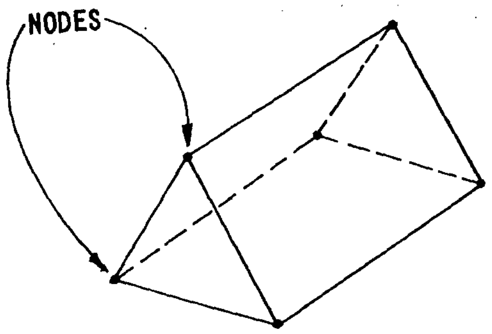
THIS TYPE OF SOLID ELEMENT HAS TWO TRIANGULAR FACES AND THREE QUADRILATERAL FACES.



TOTAL NUMBER OF DAM ELEMENT

TOTAL NUMBER OF DAM NODES =

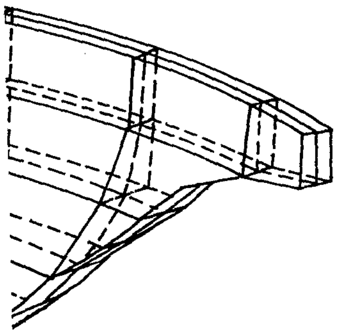
FIGURE 6-3 2-LAYER-20-NODE SOLID FINITE ELEMENT DAM MODEL (2L20ND)



TYPICAL 6-NODE PRISMATIC SOLID DAM ELEMENT

NOTE:

THIS TYPE OF SOLID ELEMENT HAS TWO TRIANGULAR FACES AND THREE QUADRILATERAL FACES.



TOTAL NUMBER OF DAM ELEMENTS = 114

TOTAL NUMBER OF DAM NODES = 768

:L2OND)

3

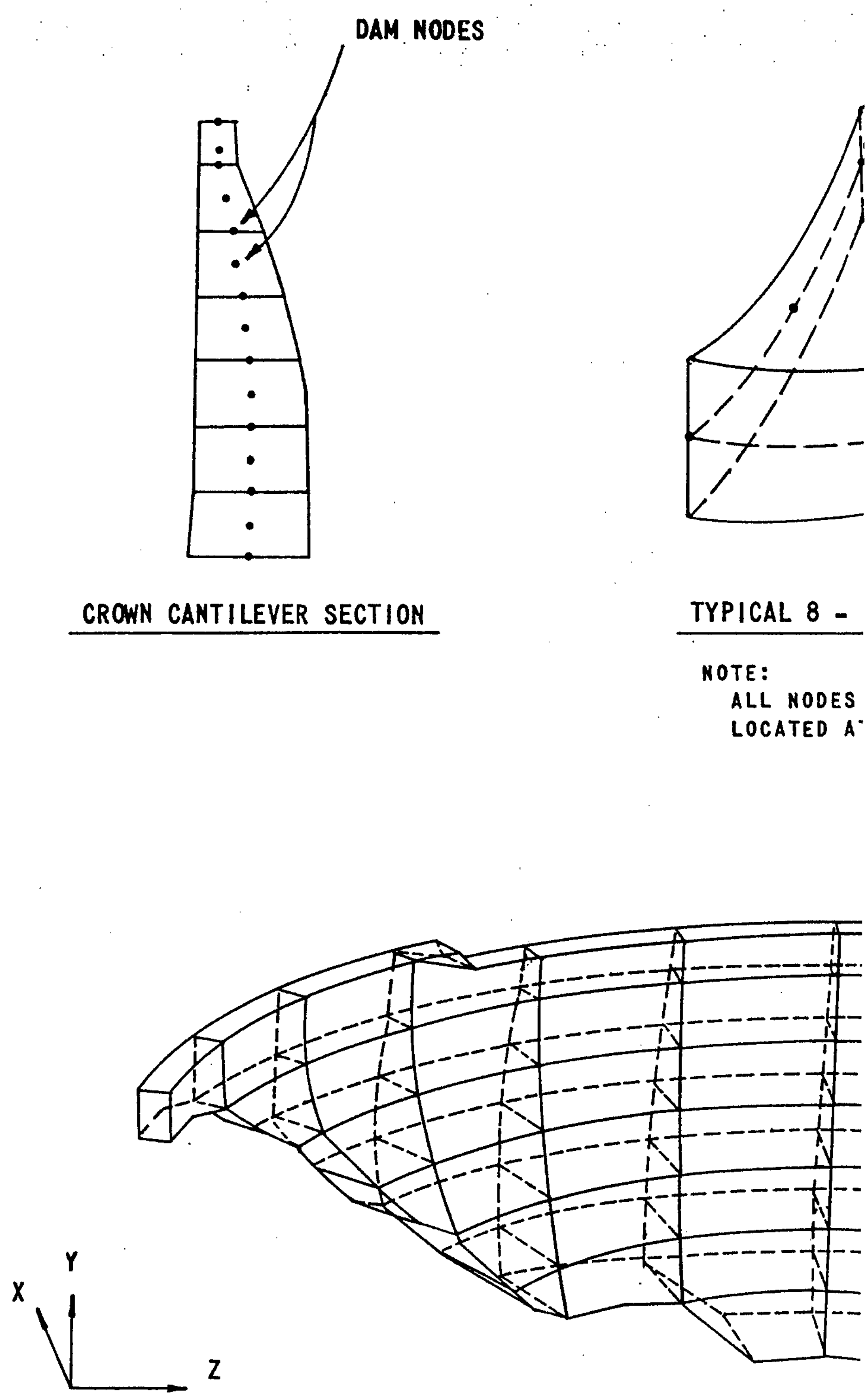
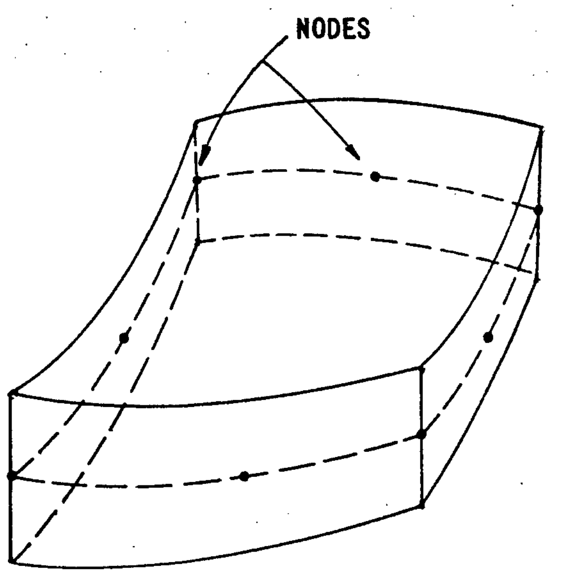
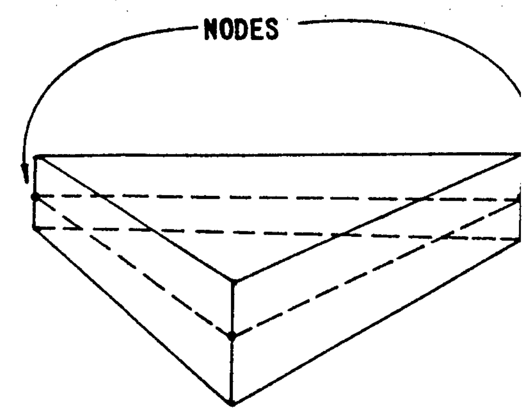


FIGURE 6-4 8-NODE THICK SHELL F



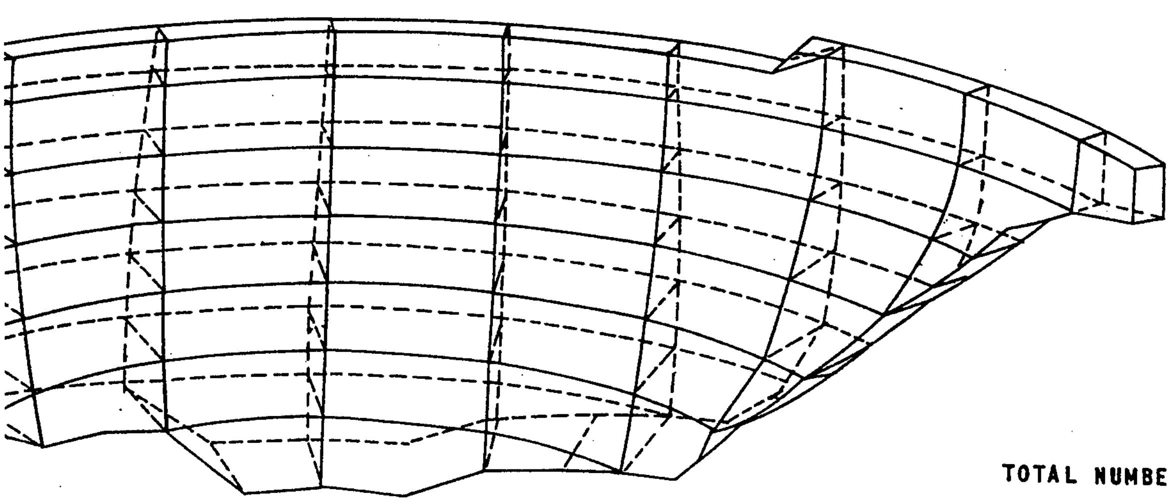
TYPICAL 8 - NODE SHELL DAM ELEMENT

NOTE:
ALL NODES OF SHELL ELEMENT ARE
LOCATED AT THE MID-SURFACE.



TYPICAL 3 - NODE PRISMATIC SHELL DA

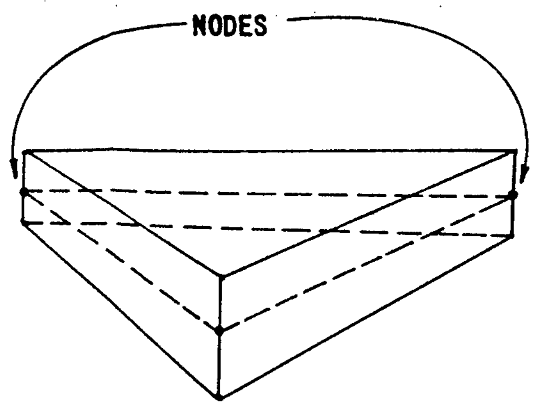
NOTE:
ALL NODES OF SHELL ELEMENT AR
LOCATED AT THE MID-SURFACE.



TOTAL NUMBER OF DAM ELEMENTS = 57

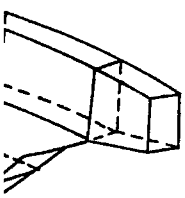
TOTAL NUMBER OF DAM NODES = 206

-4 8-NODE THICK SHELL FINITE ELEMENT DAM MODEL (8NTSD)



TYPICAL 3 - NODE PRISMATIC SHELL DAM ELEMENT

NOTE:
ALL NODES OF SHELL ELEMENT ARE LOCATED AT THE MID-SURFACE.



TOTAL NUMBER OF DAM ELEMENTS = 57

TOTAL NUMBER OF DAM NODES = 206

3

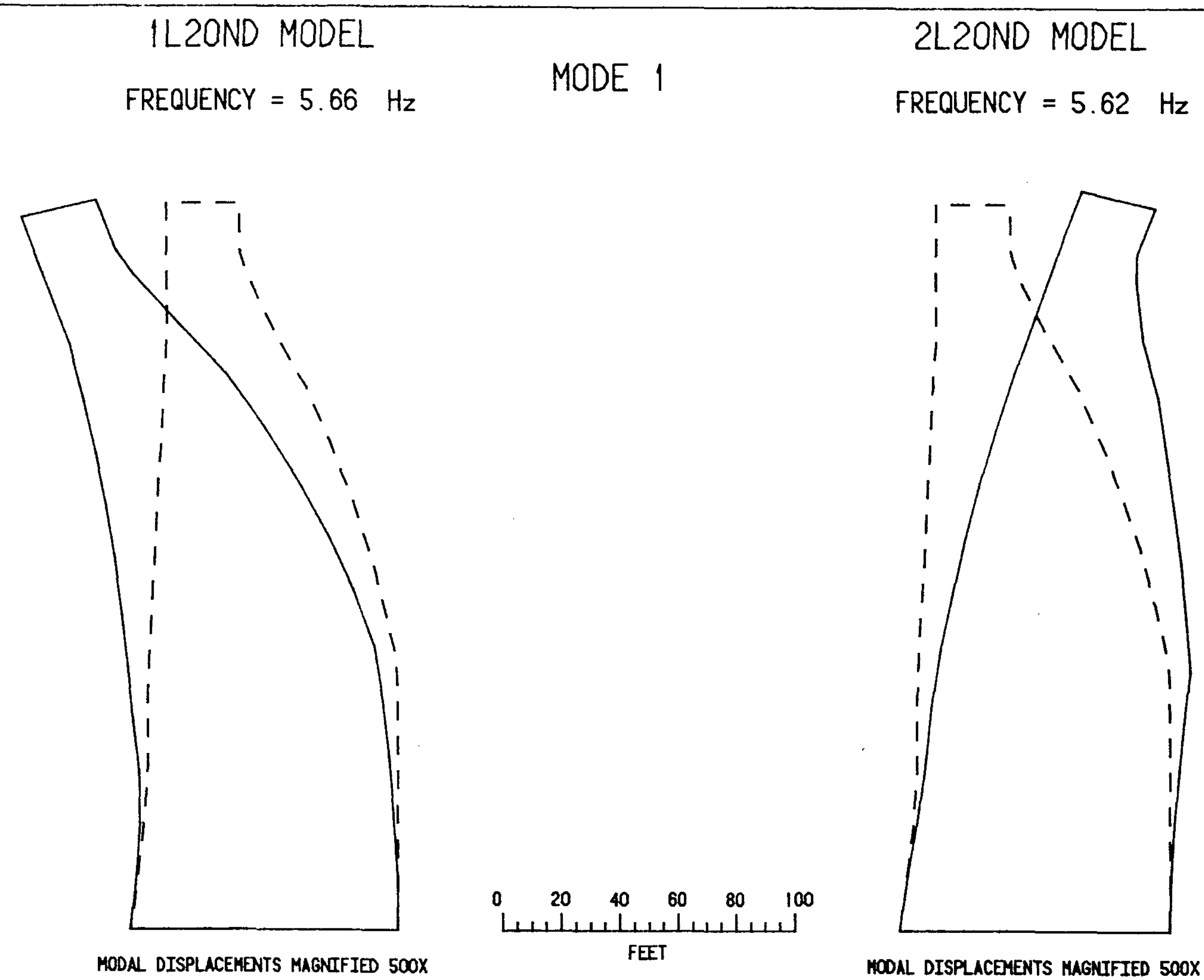


FIGURE 6-5 PLOT OF MODE SHAPES OF CROWN CANTILEVER

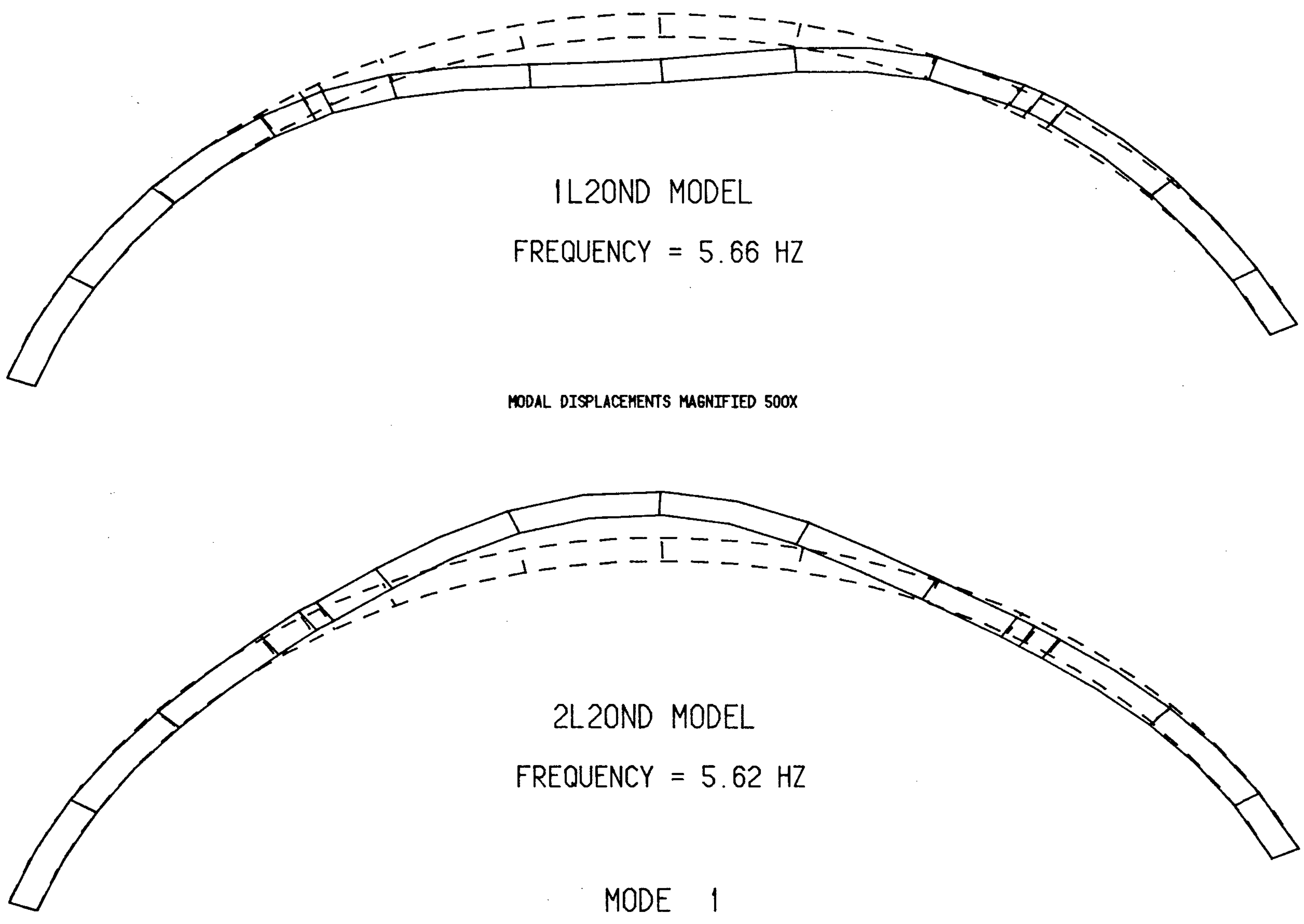


FIGURE 6-6 PLOT OF MODE SHAPES OF CREST

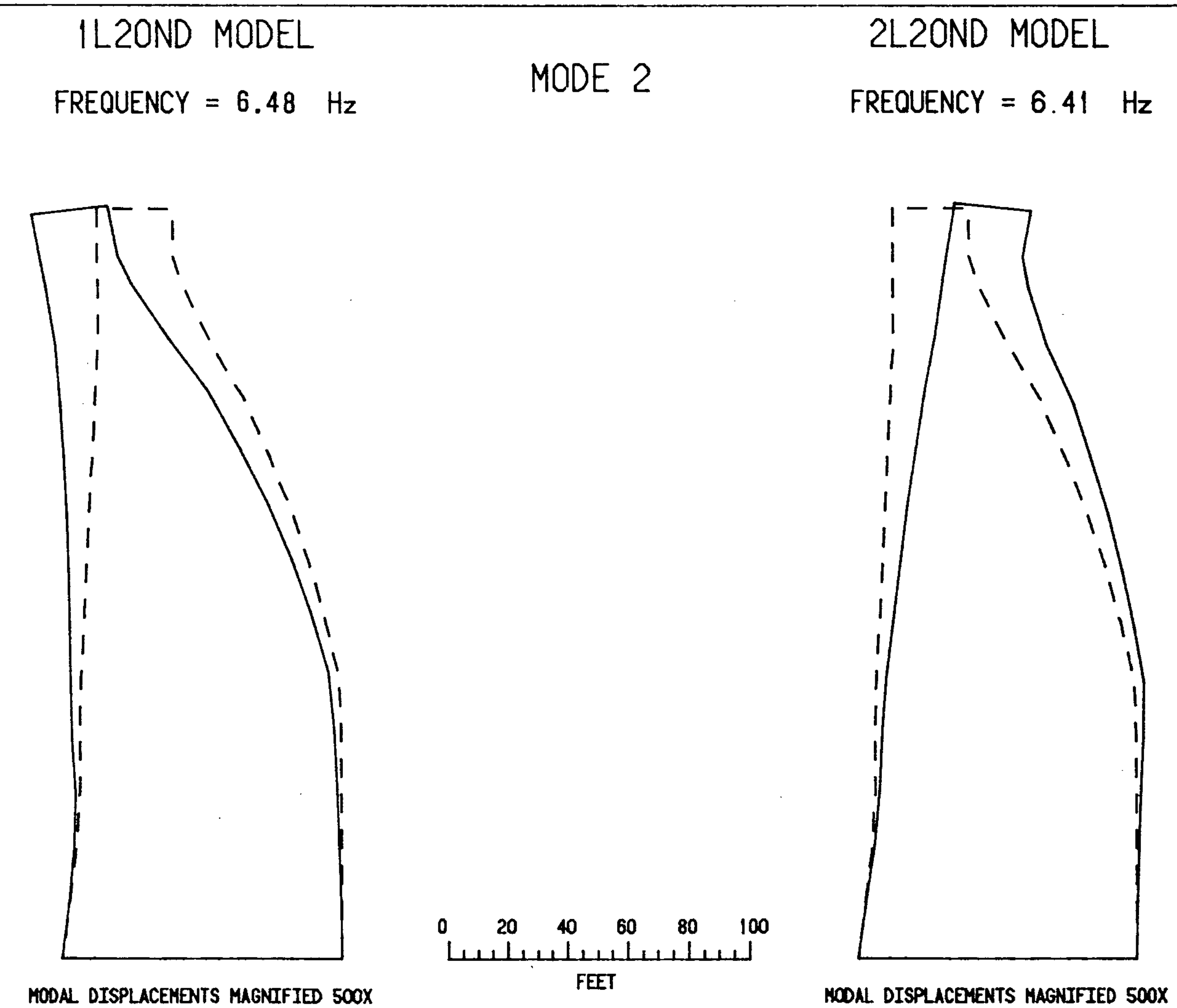


FIGURE 6-7 PLOT OF MODE SHAPES OF CROWN CANTILEVER

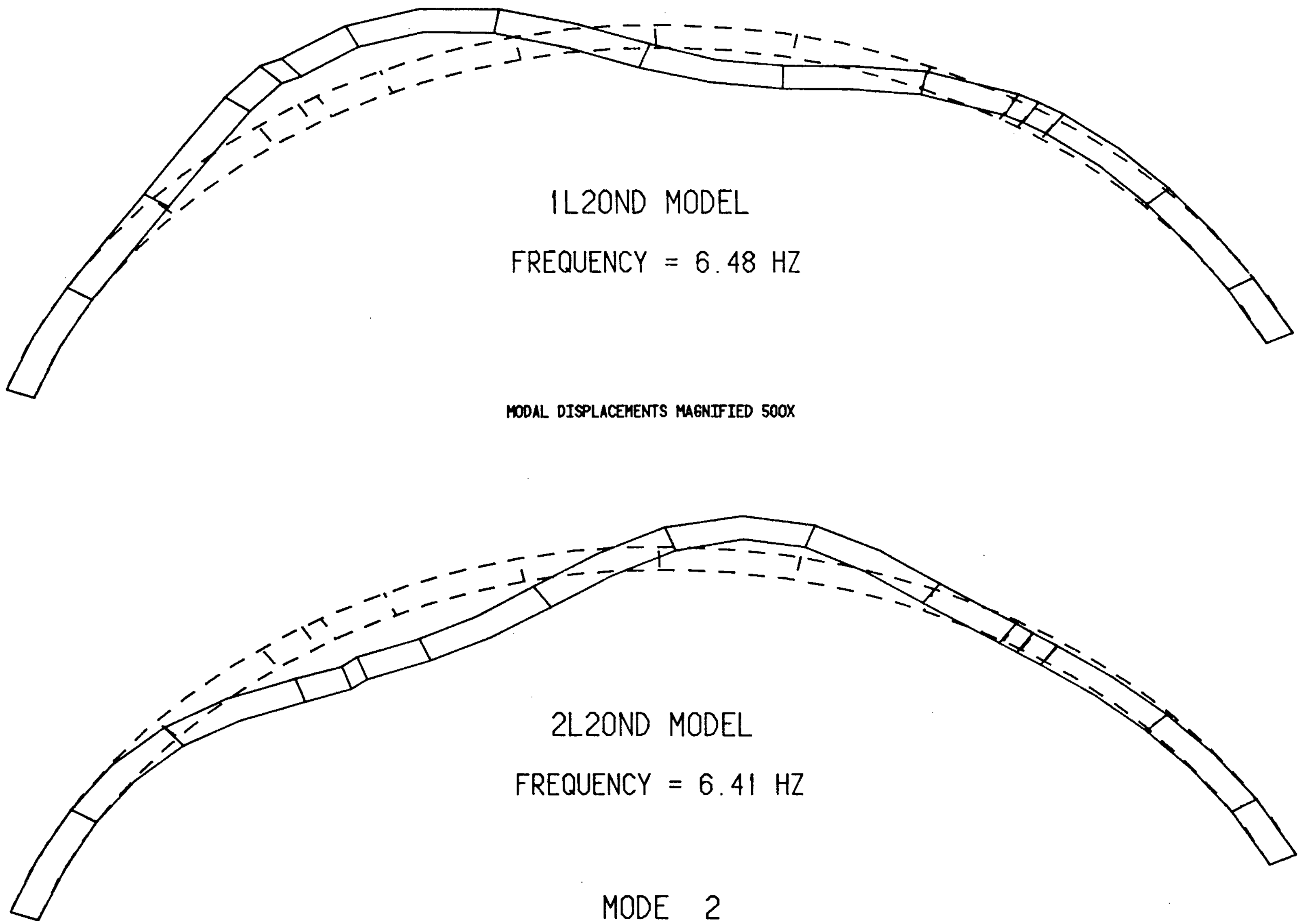


FIGURE 6-8 PLOT OF MODE SHAPES OF CREST

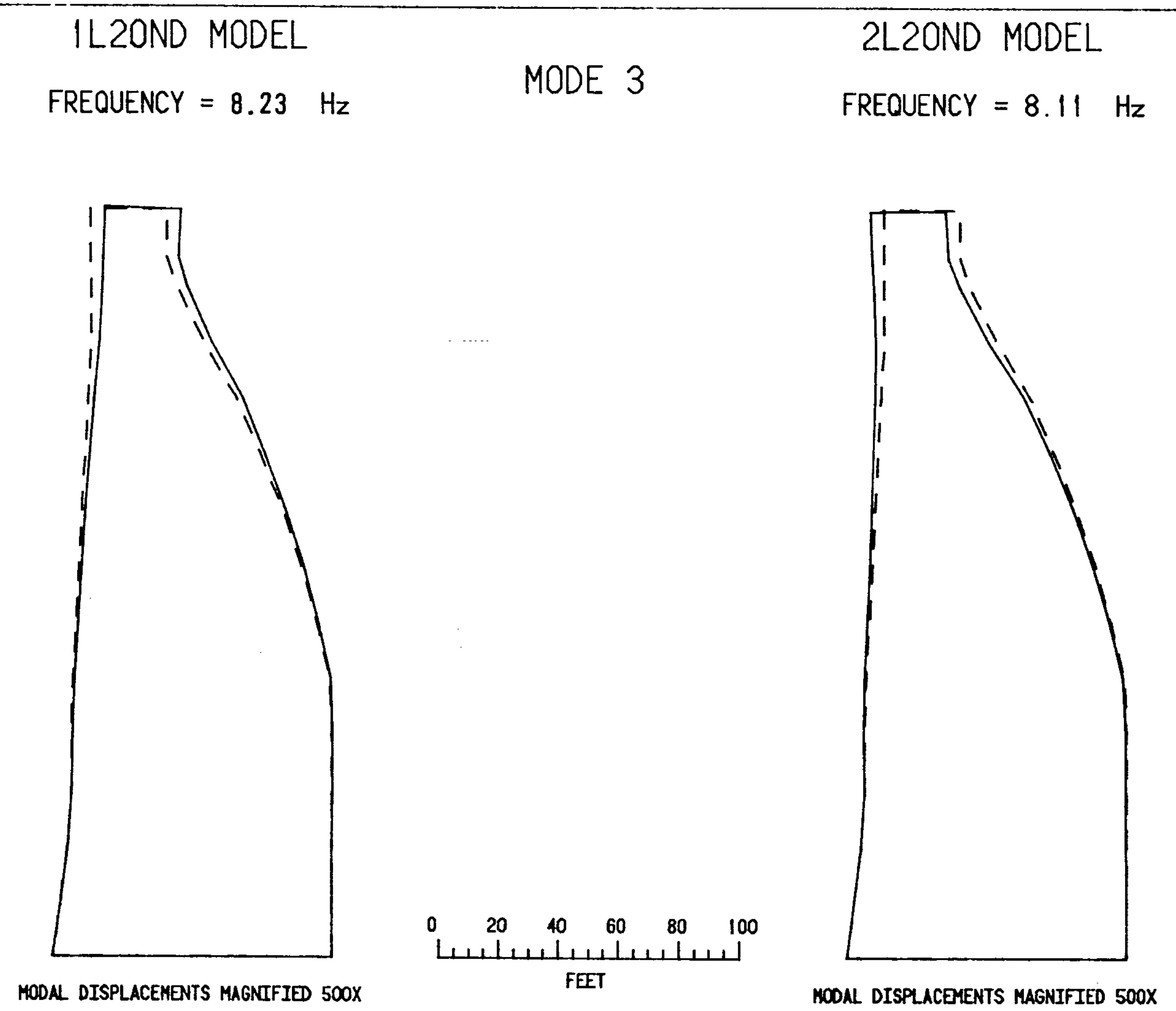


FIGURE 6-9 PLOT OF MODE SHAPES OF CROWN CANTILEVER

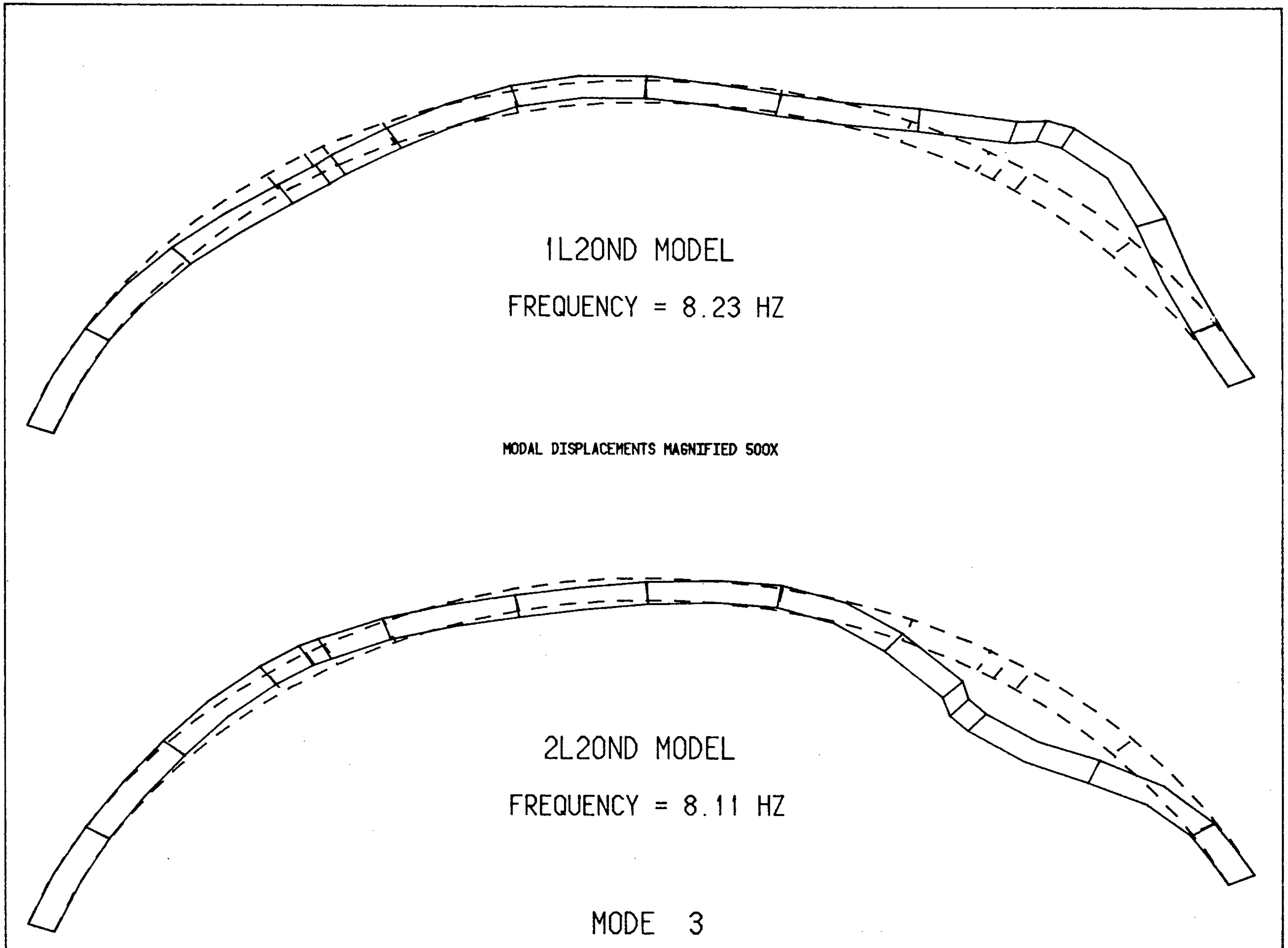


FIGURE 6-10 PLOT OF MODE SHAPES OF CREST

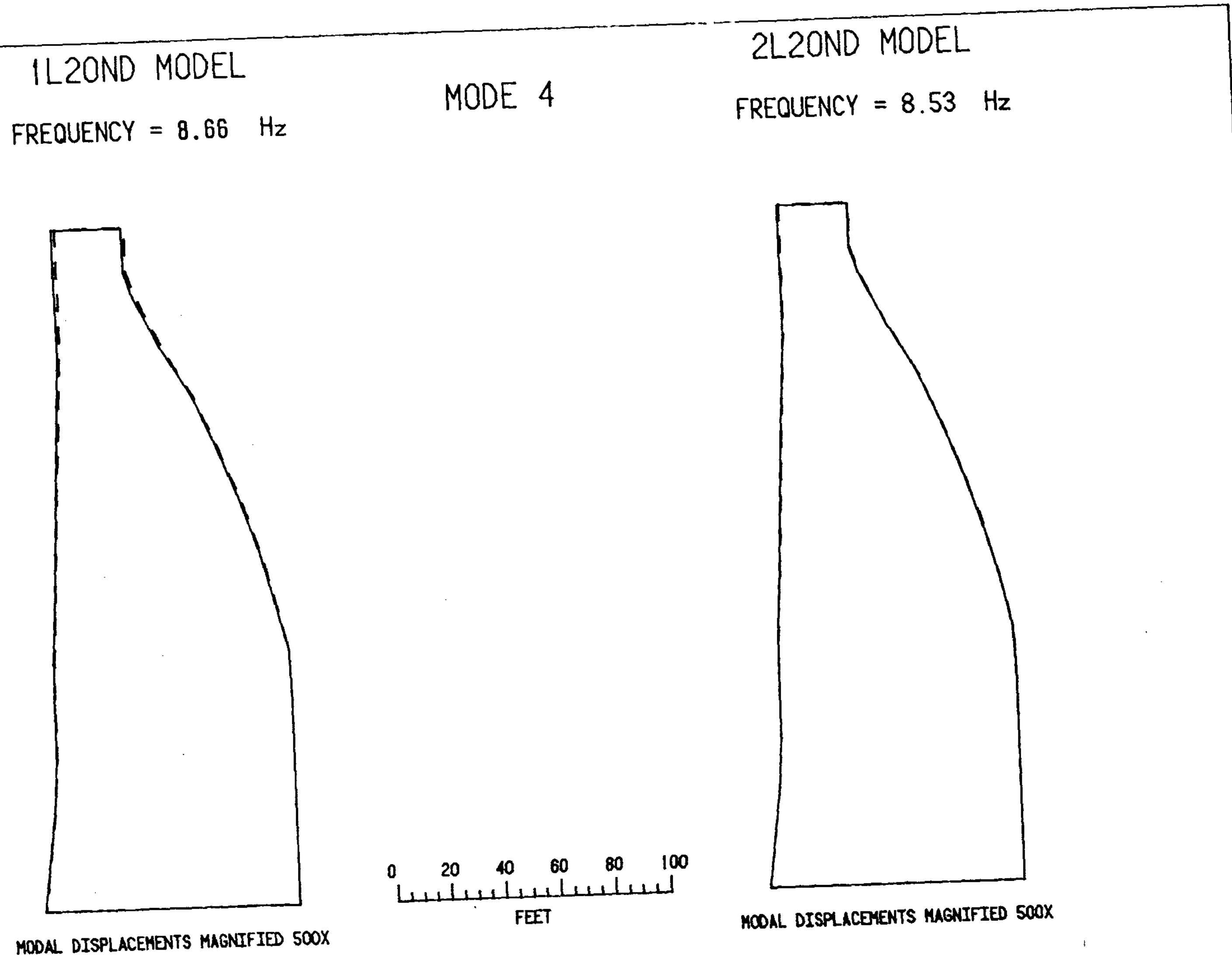


FIGURE 6-11 PLOT OF MODE SHAPES OF CROWN CANTILEVER

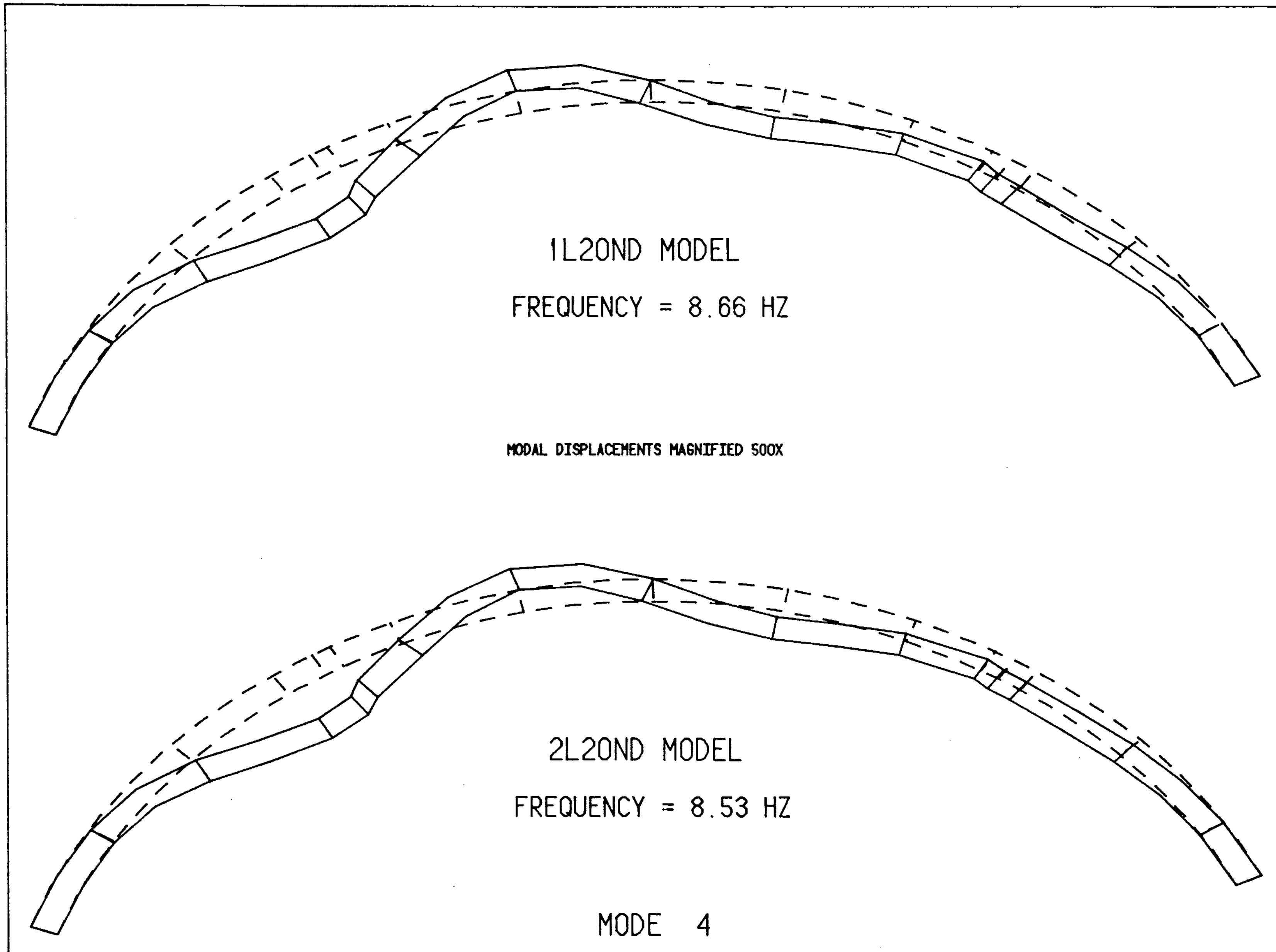


FIGURE 6-12 PLOT OF MODE SHAPES OF CREST

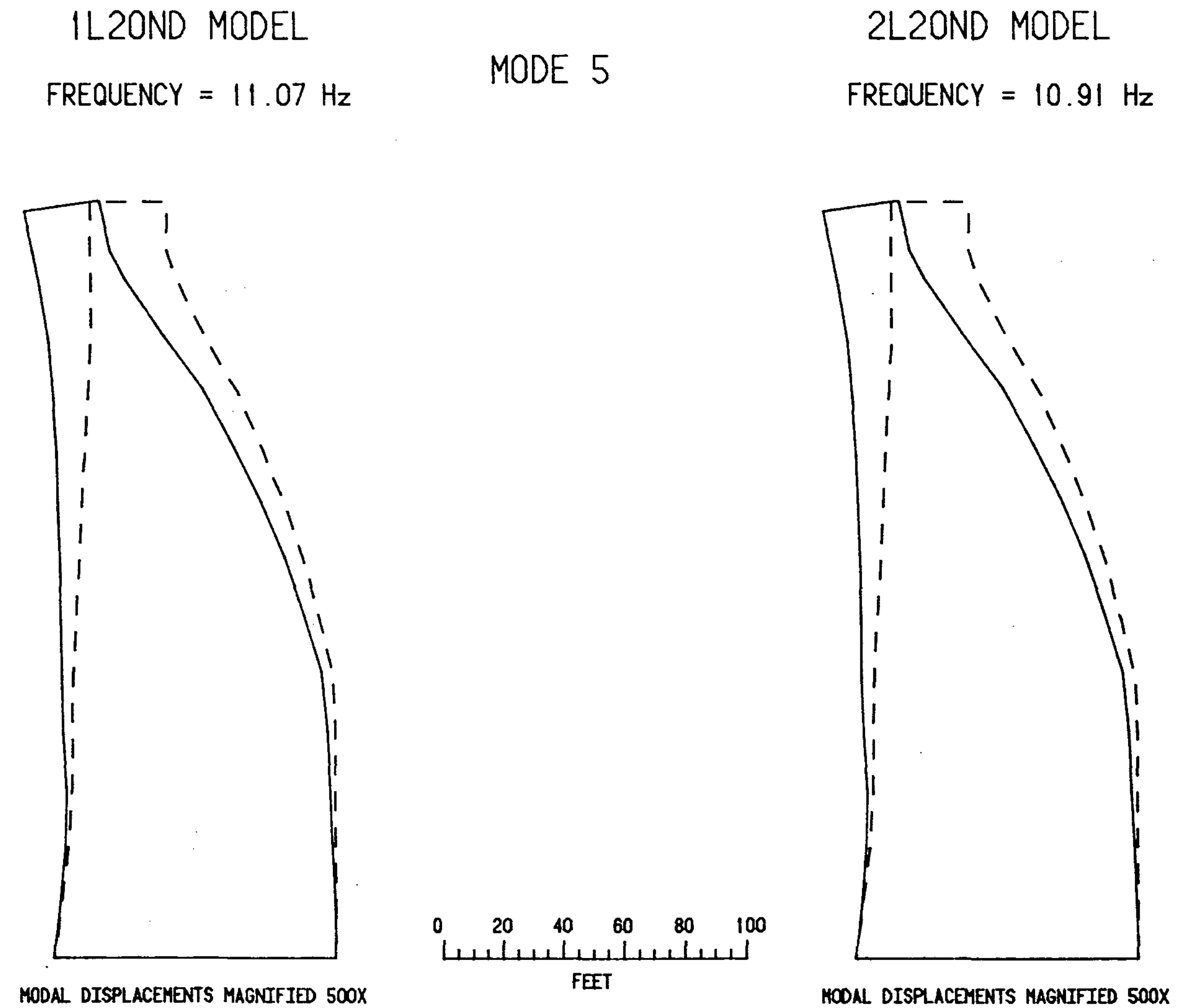


FIGURE 6-13 PLOT OF MODE SHAPES OF CROWN CANTILEVER

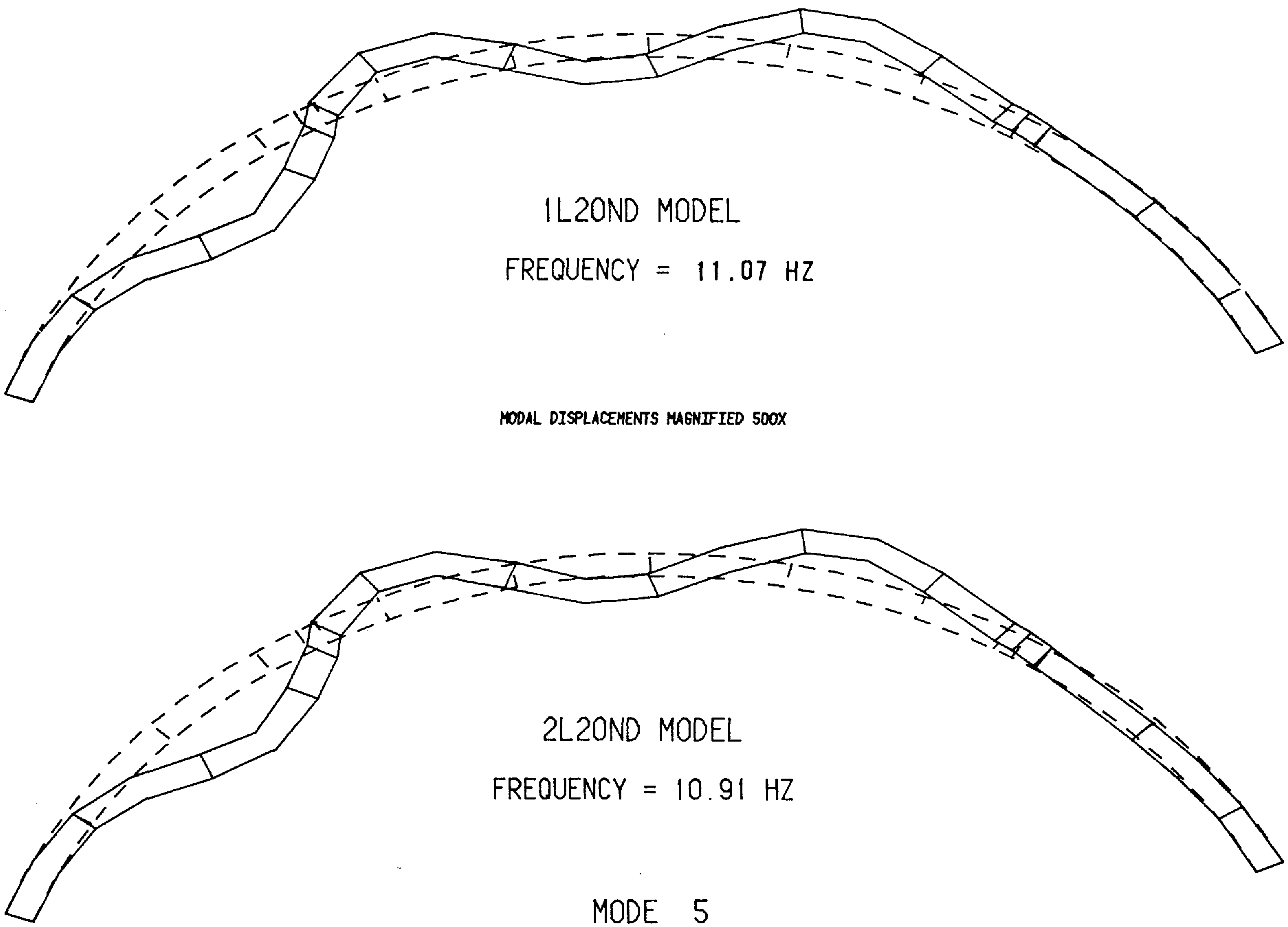


FIGURE 6-14 PLOT OF MODE SHAPES OF CREST

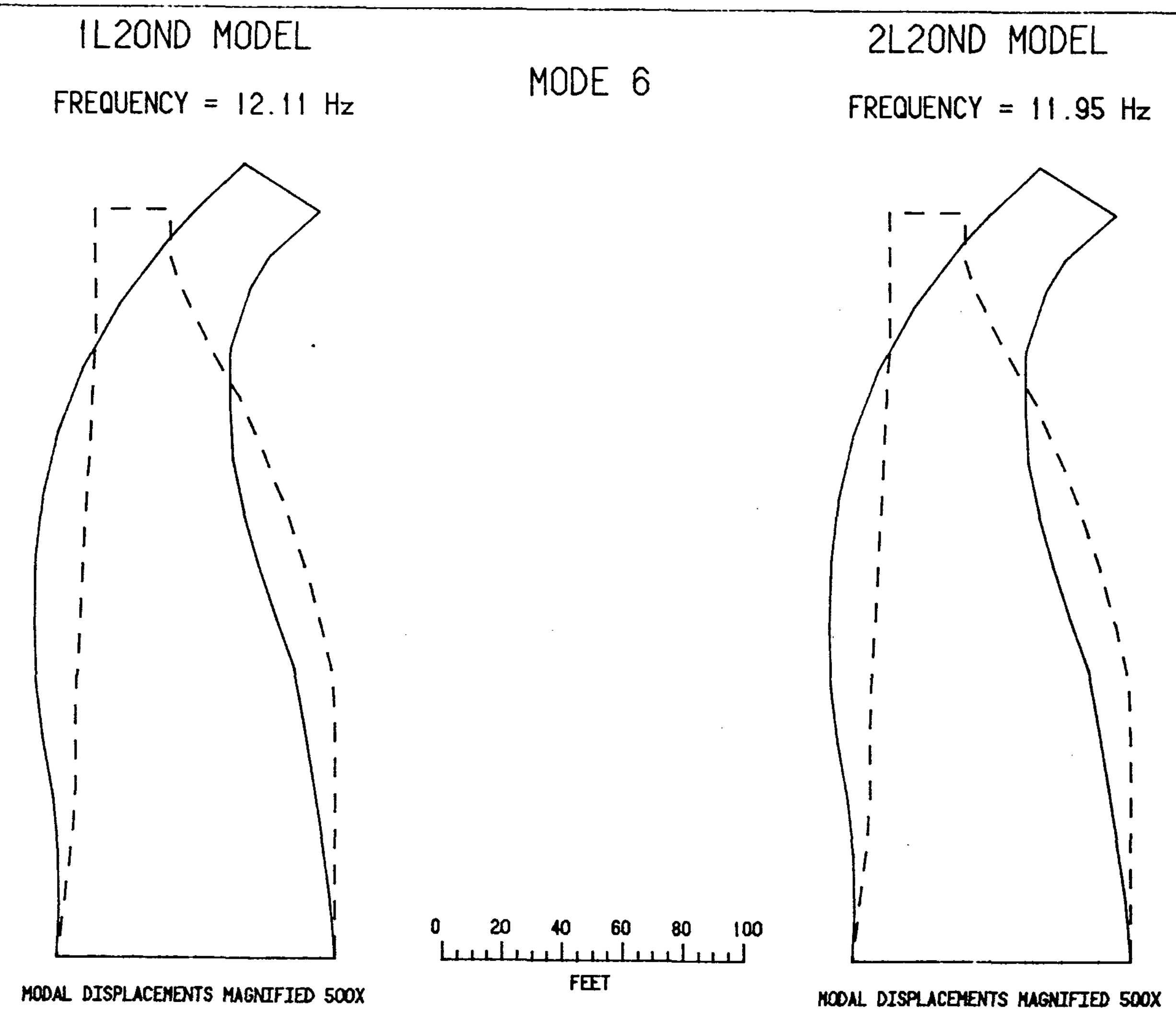


FIGURE 6-15 PLOT OF MODE SHAPES OF CROWN CANTILEVER

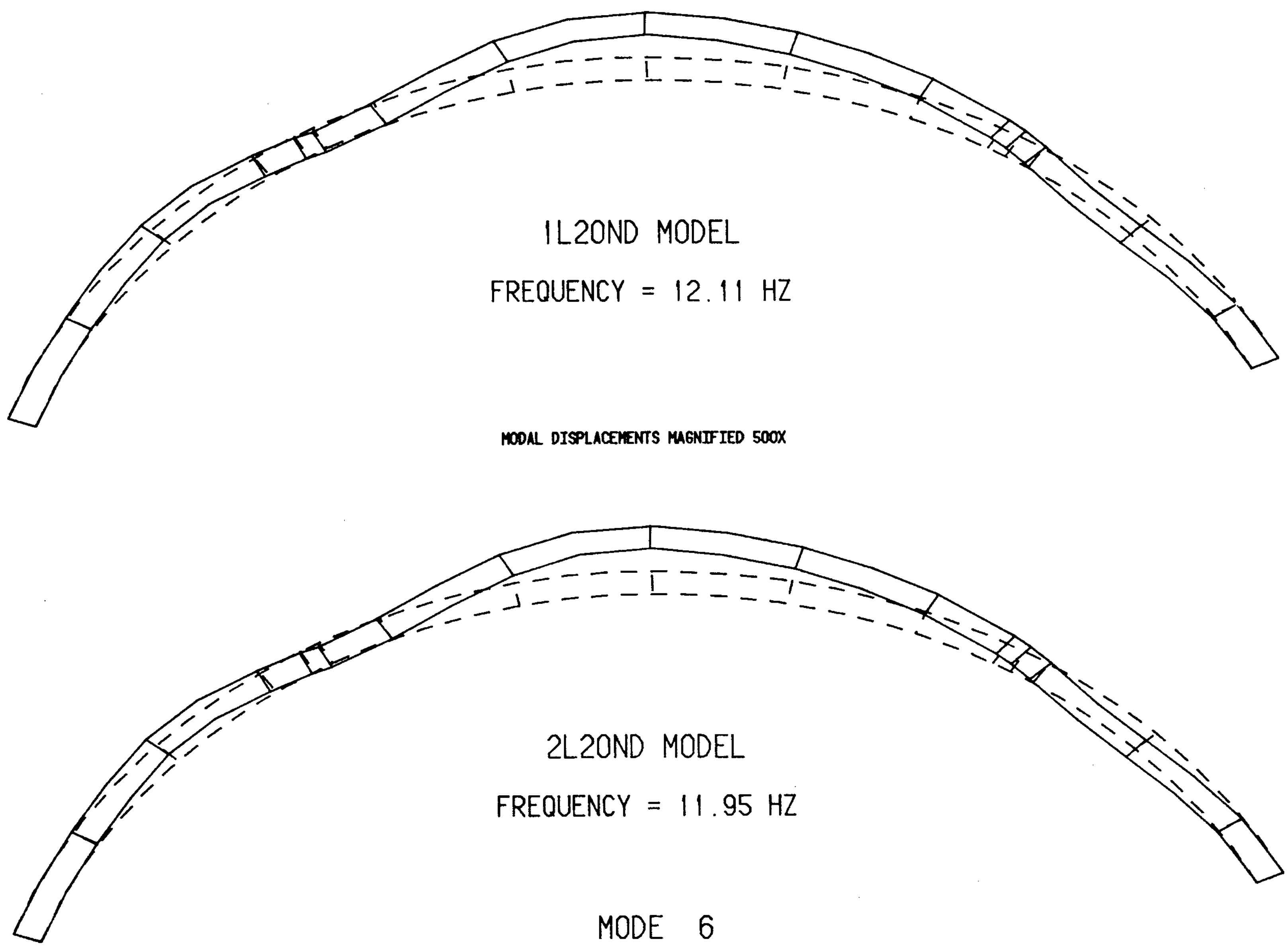


FIGURE 6-16 PLOT OF MODE SHAPES OF CREST

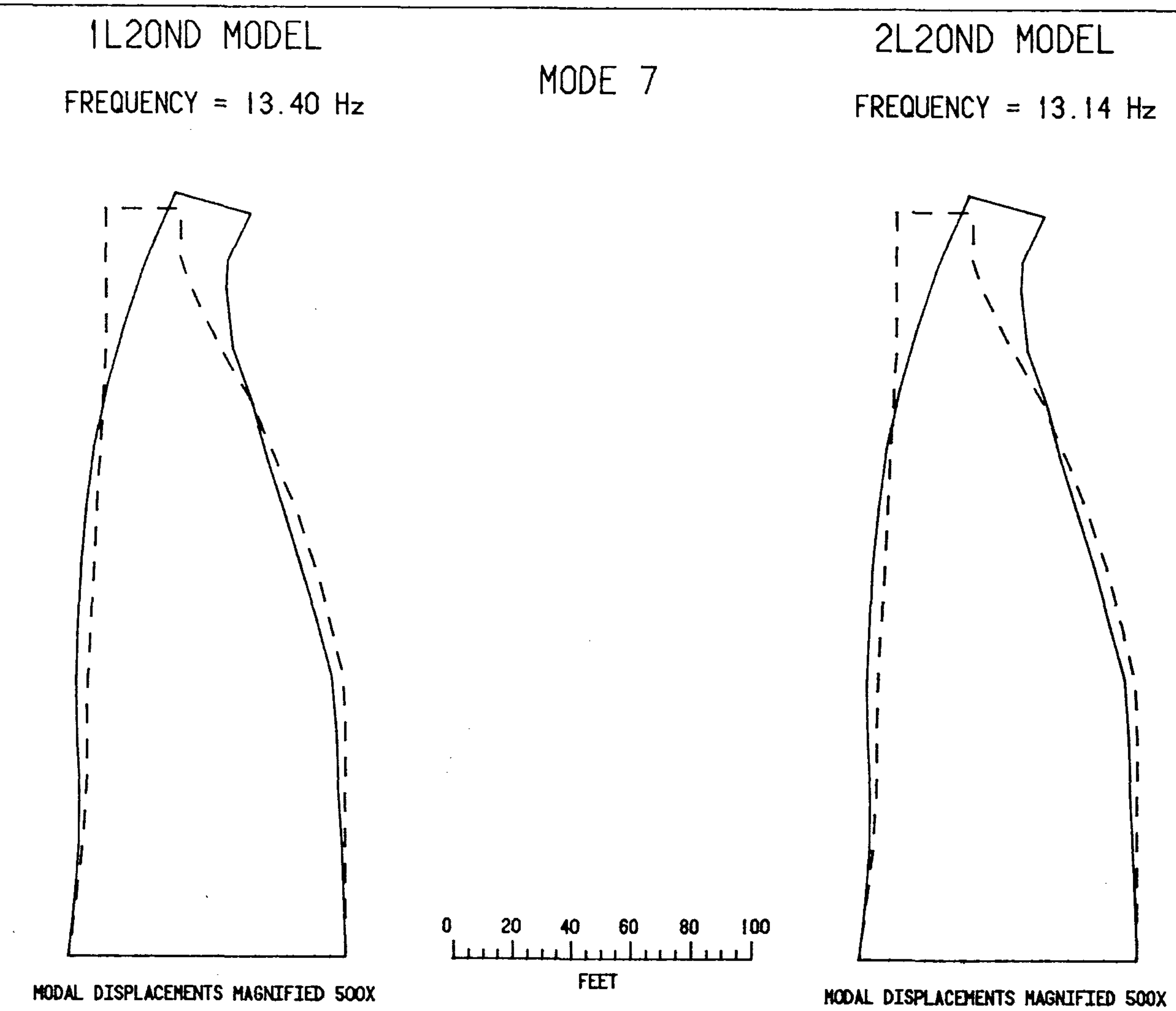


FIGURE 6-17 PLOT OF MODE SHAPES OF CROWN CANTILEVER

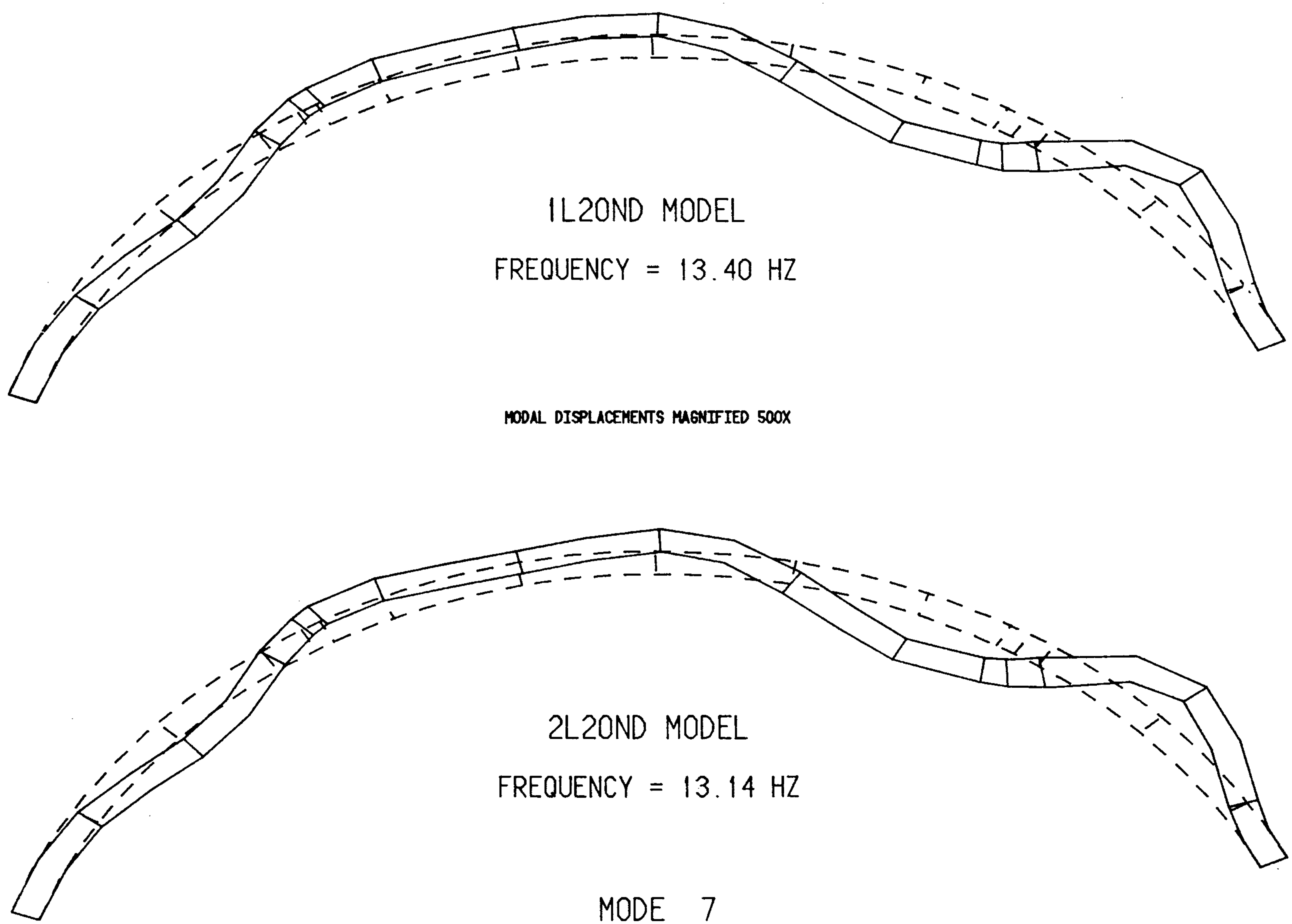


FIGURE 6-18 PLOT OF MODE SHAPES OF CREST

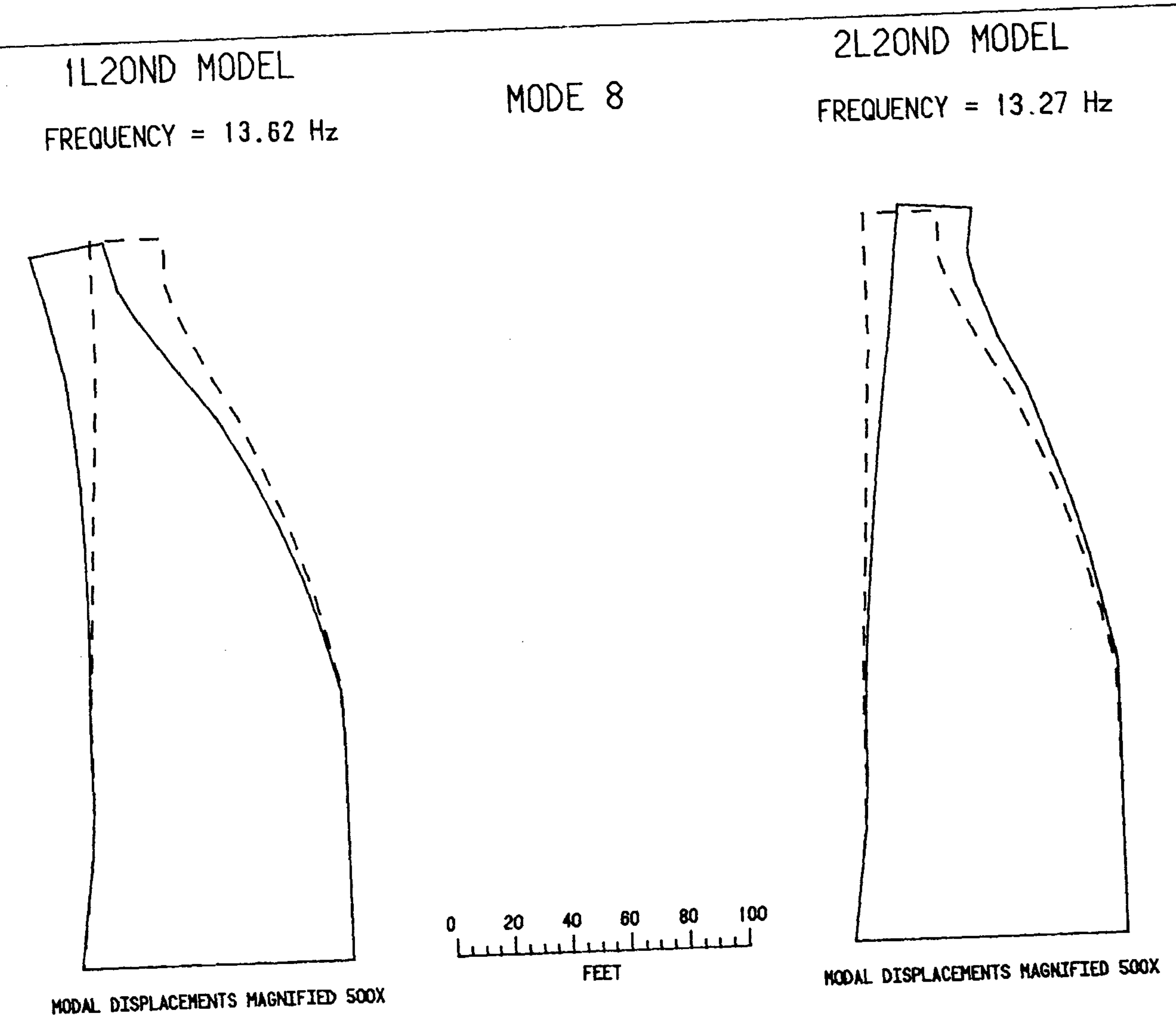


FIGURE 6-19 PLOT OF MODE SHAPES OF CROWN CANTILEVER

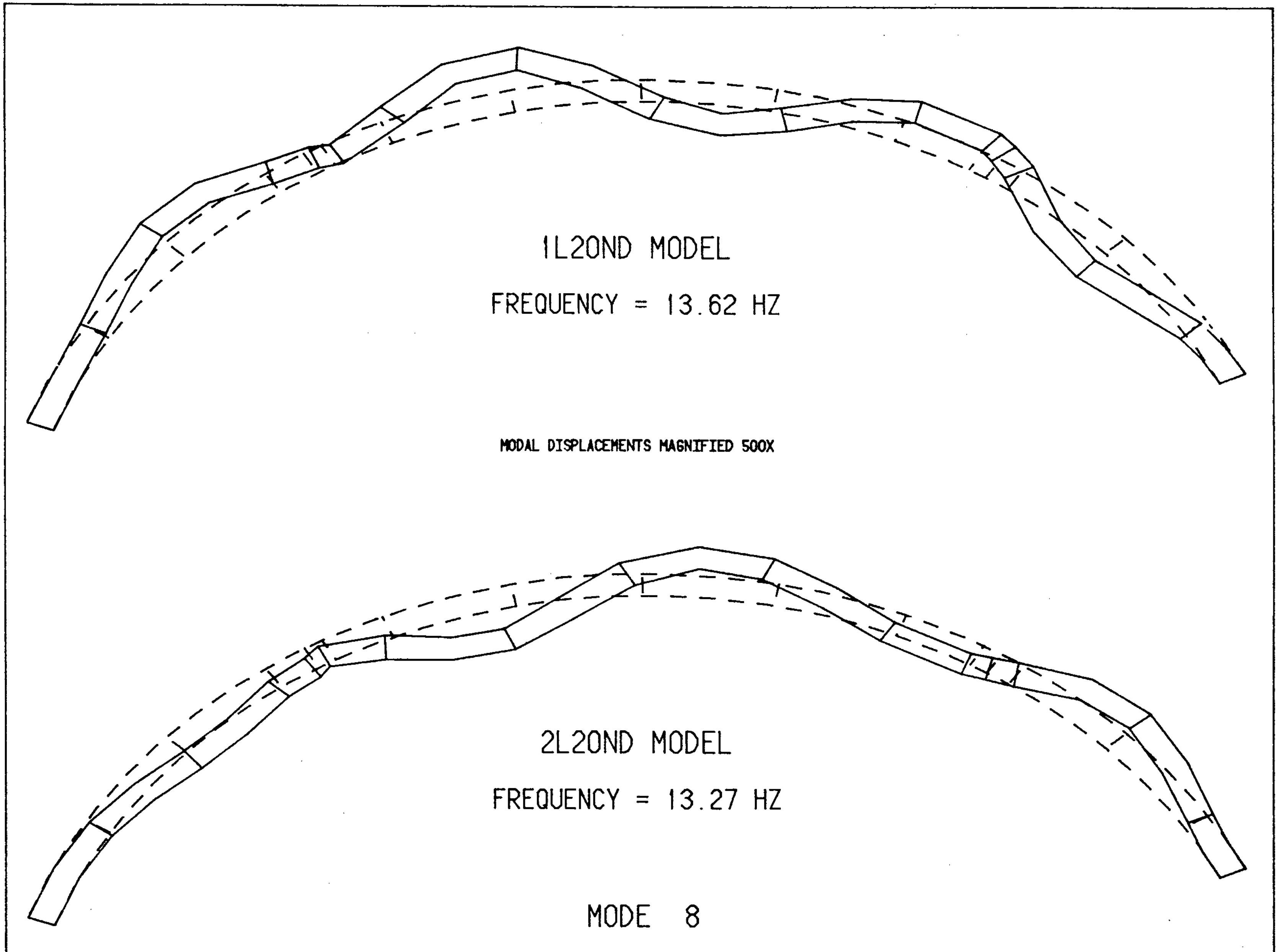


FIGURE 6-20 PLOT OF MODE SHAPES OF CREST

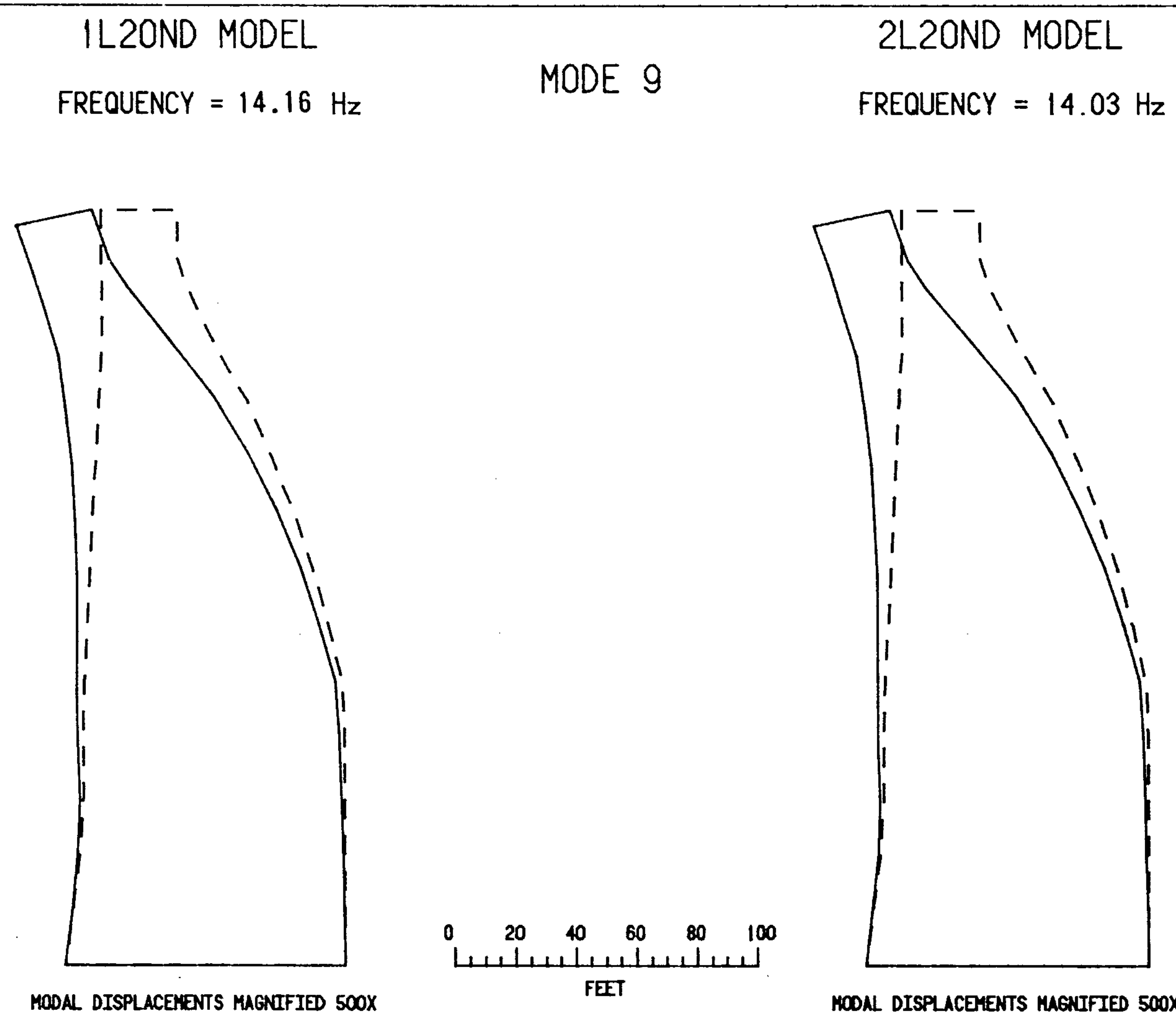


FIGURE 6-21 PLOT OF MODE SHAPES OF CROWN CANTILEVER

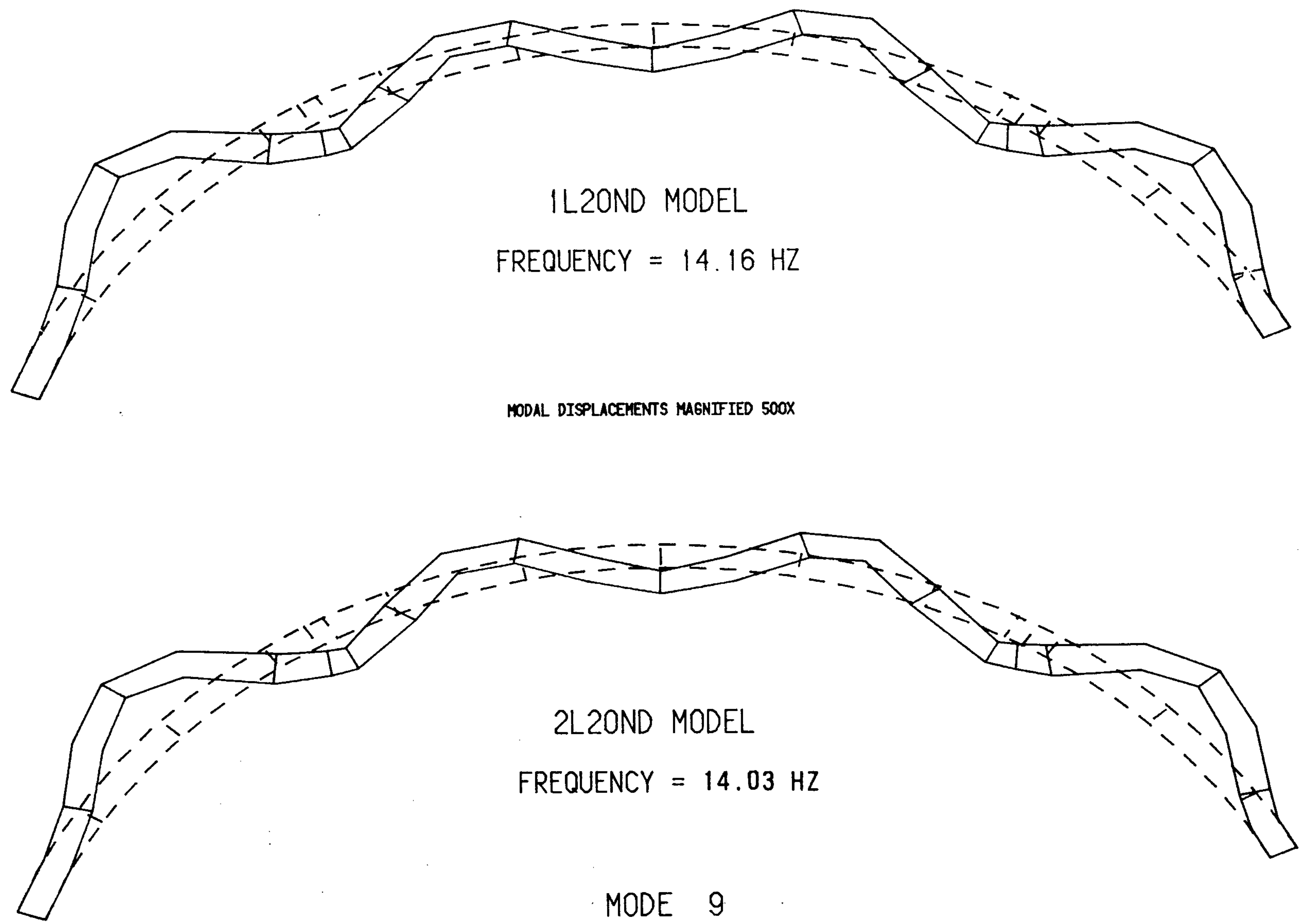


FIGURE 6-22 PLOT OF MODE SHAPES OF CREST

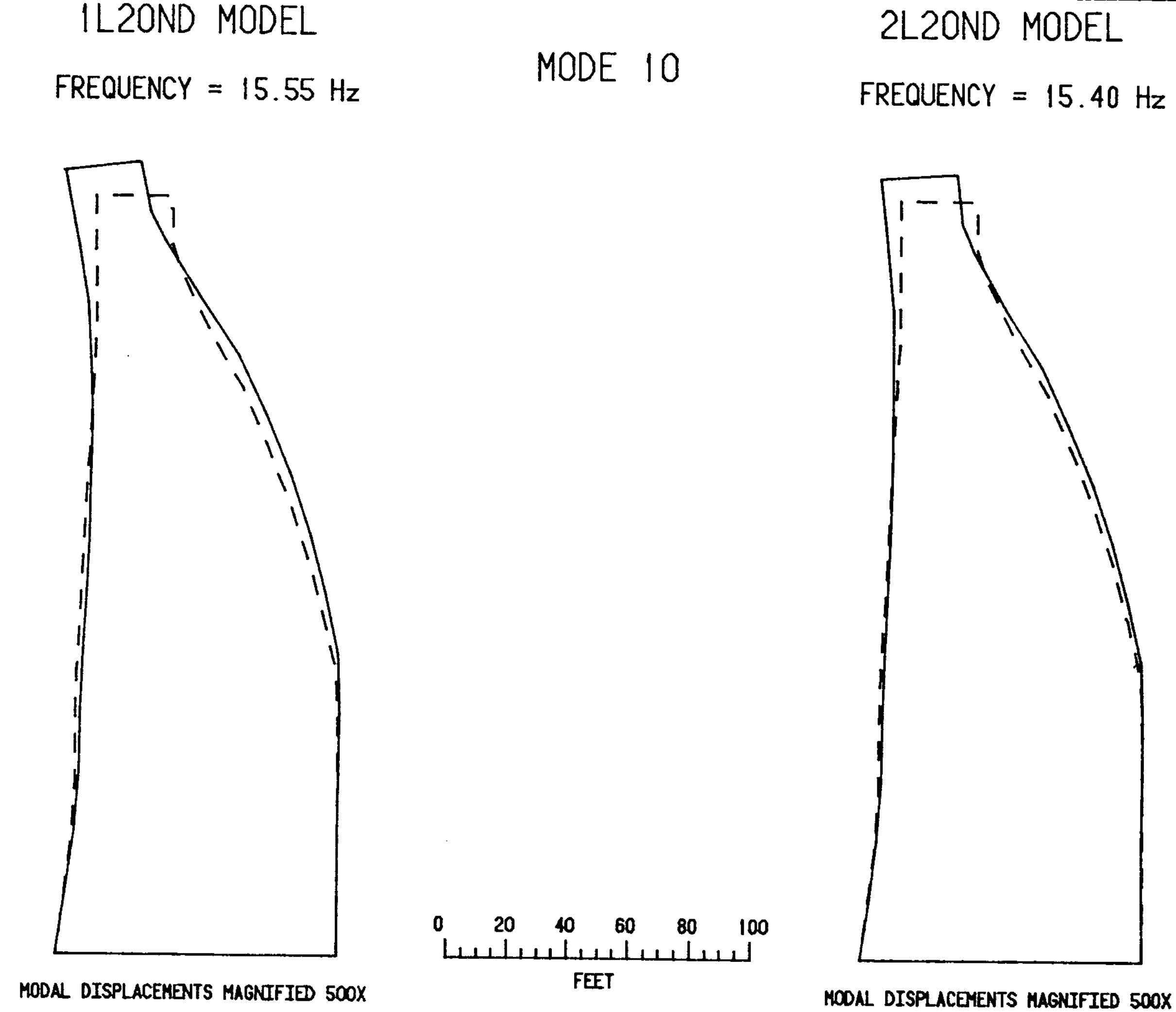


FIGURE 6-23 PLOT OF MODE SHAPES OF CROWN CANTILEVER

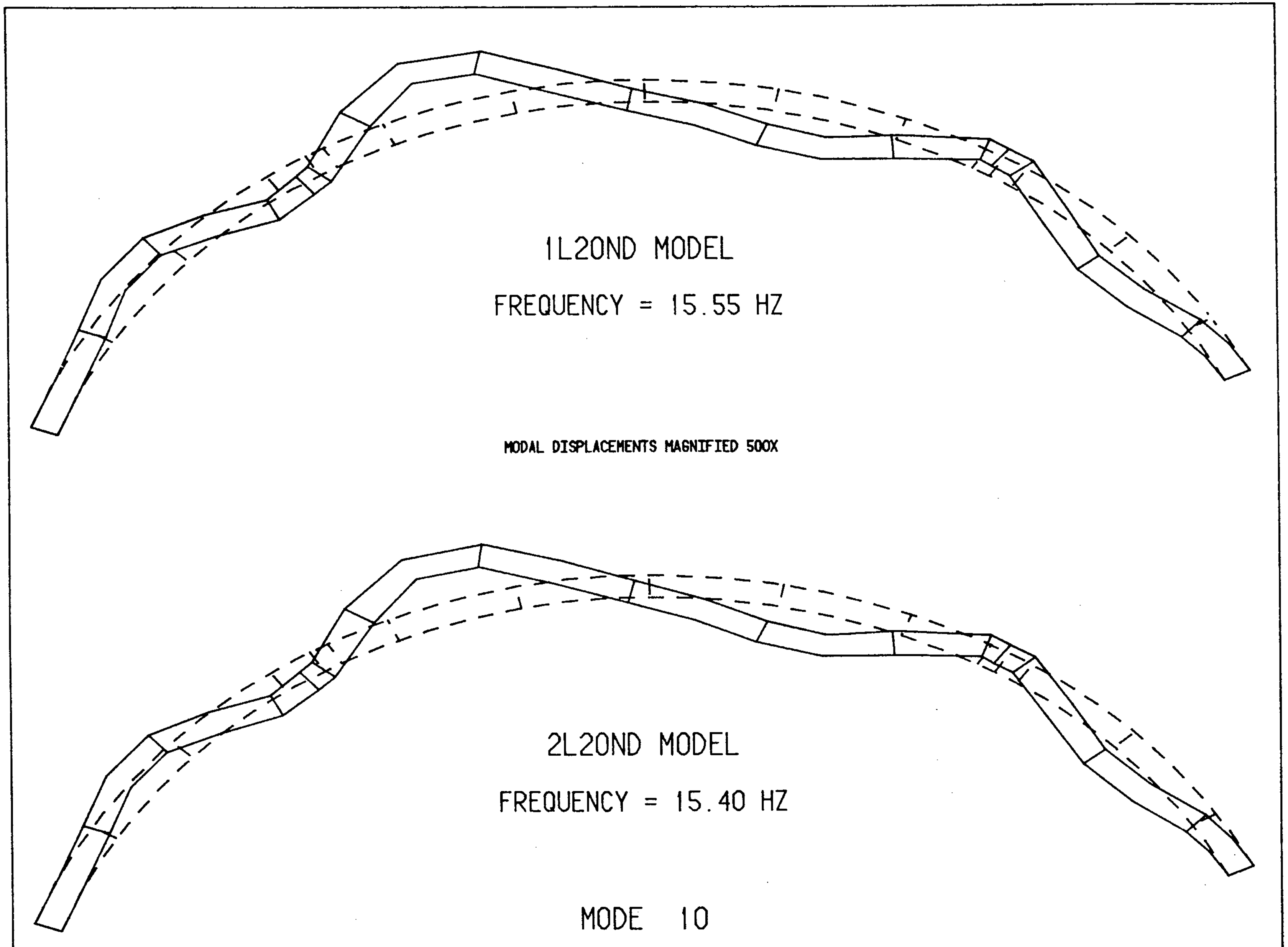


FIGURE 6-24 PLOT OF MODE SHAPES OF CREST

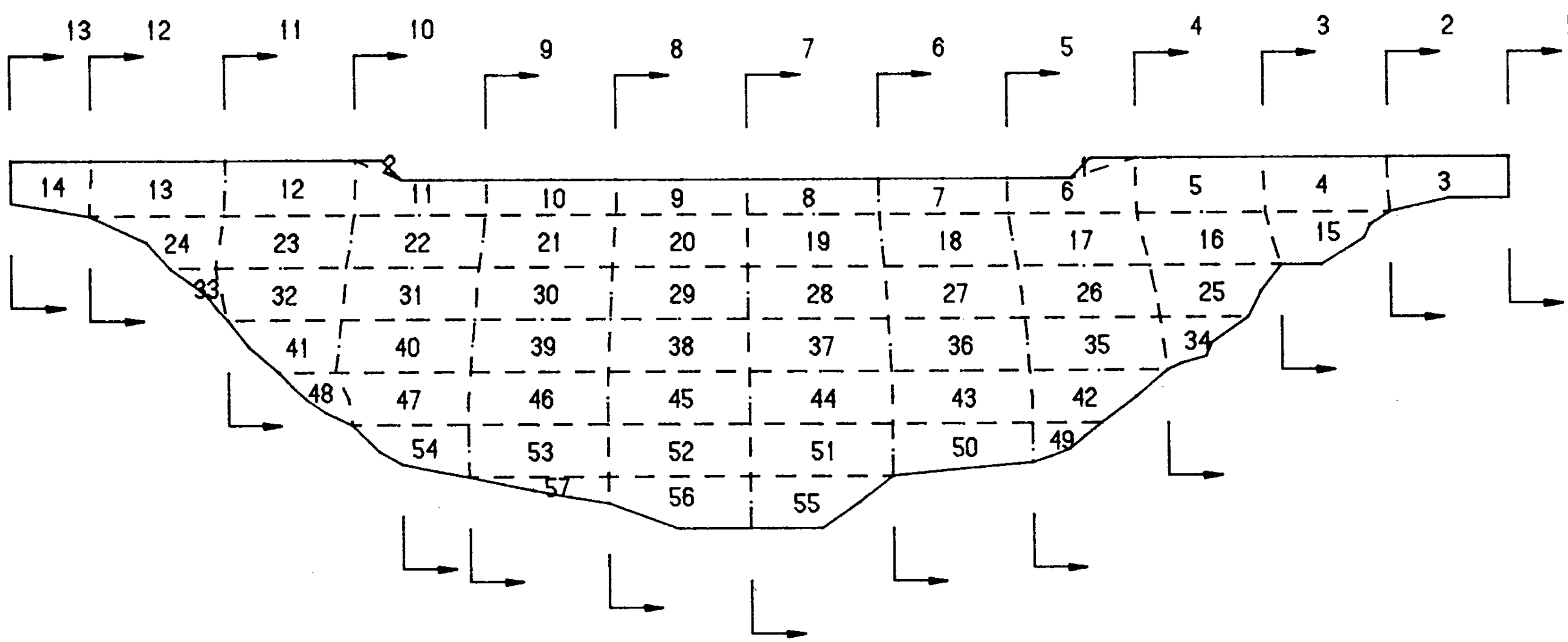


FIGURE 6-25 DEVELOPED VIEW OF UPSTREAM FACE, LOOKING DOWNSTREAM
ELEMENT NUMBERS ARE SHOWN

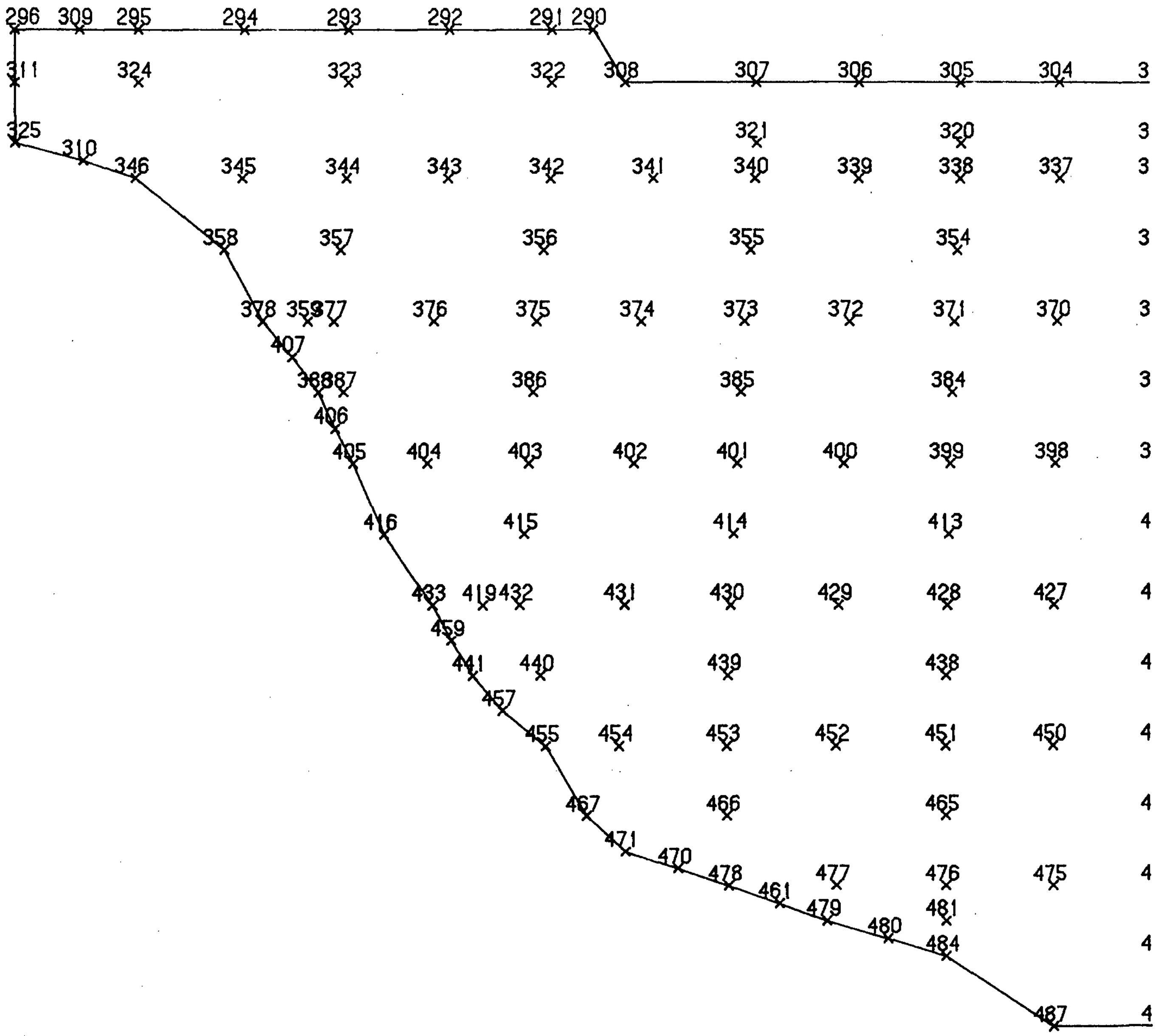


FIGURE 6-26 DEVELOPED VIEW O

NODE NU

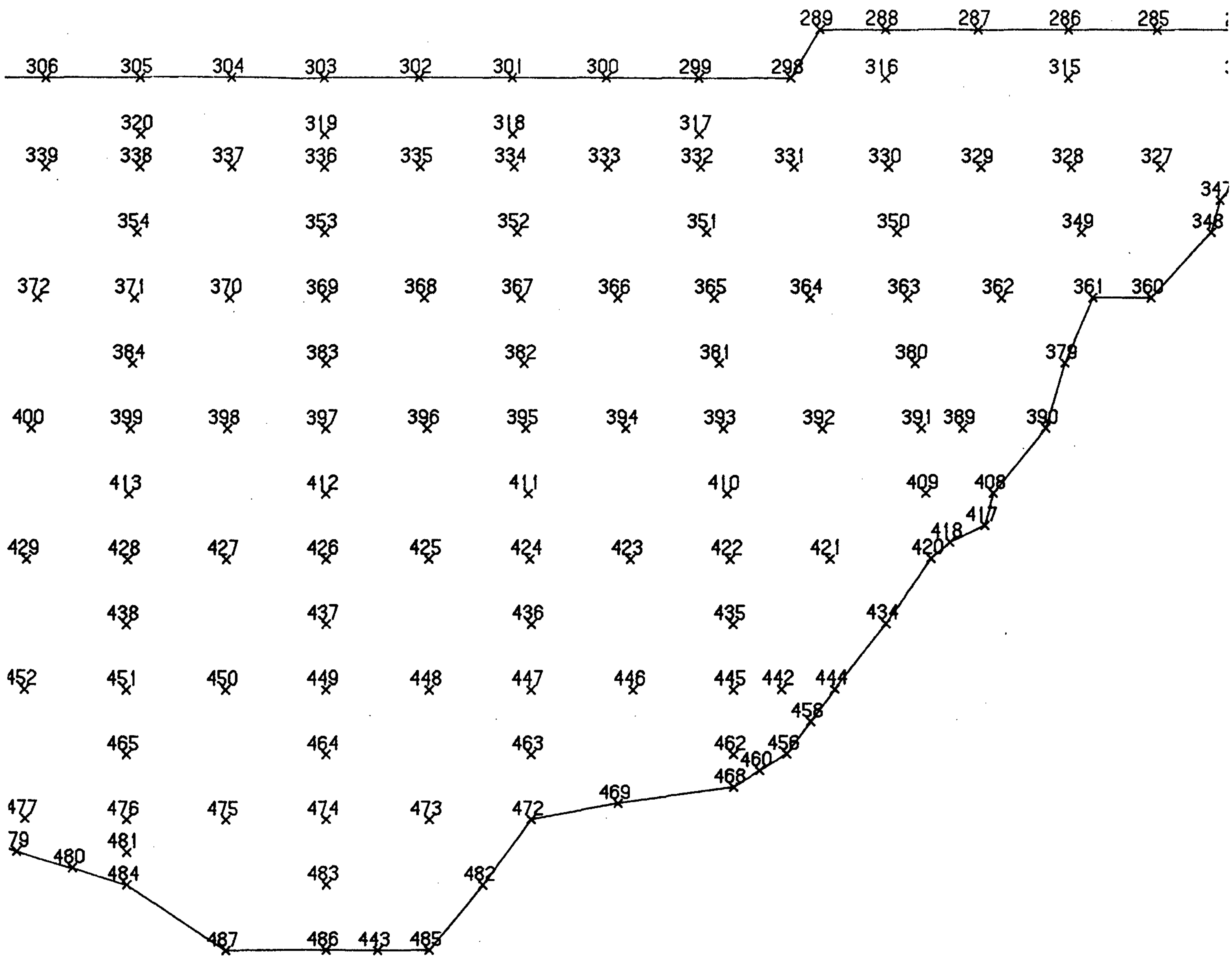
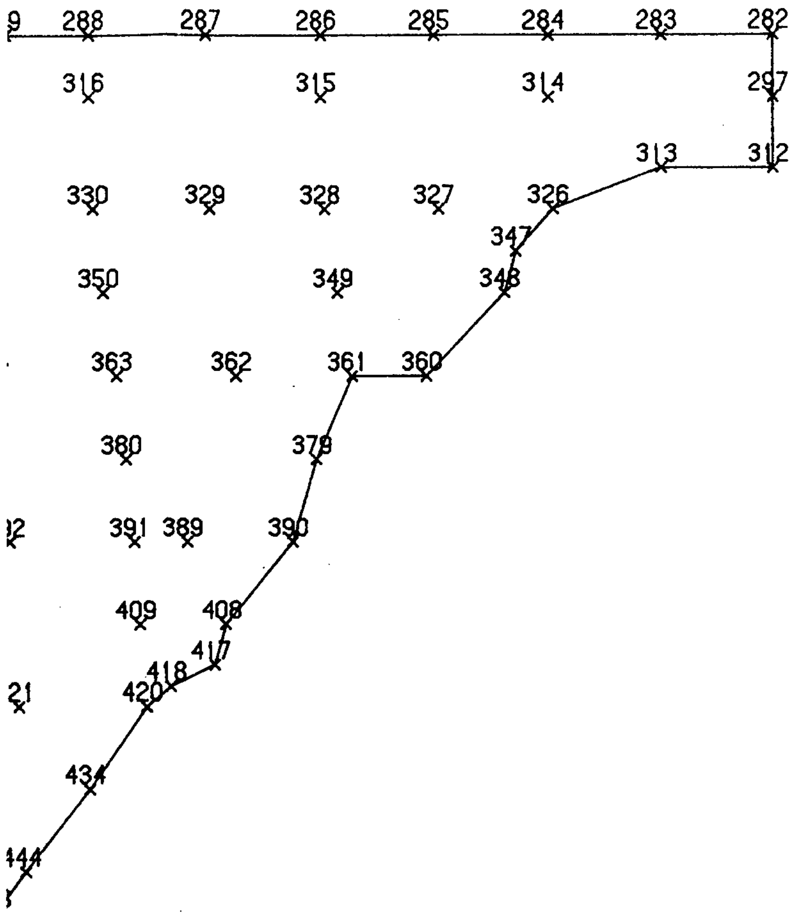


FIGURE 6-26 DEVELOPED VIEW OF UPSTREAM FACE, LOOKING DOWNSTREAM

NODE NUMBERS ARE SHOWN


9 288 287 286 285 284 283 282
316 315 314 297
313 312
330 329 328 327 326
347
350 349 348
363 362 361 360
380 379
12 391 389 390
409 408
417
418
21 420
434
444

3

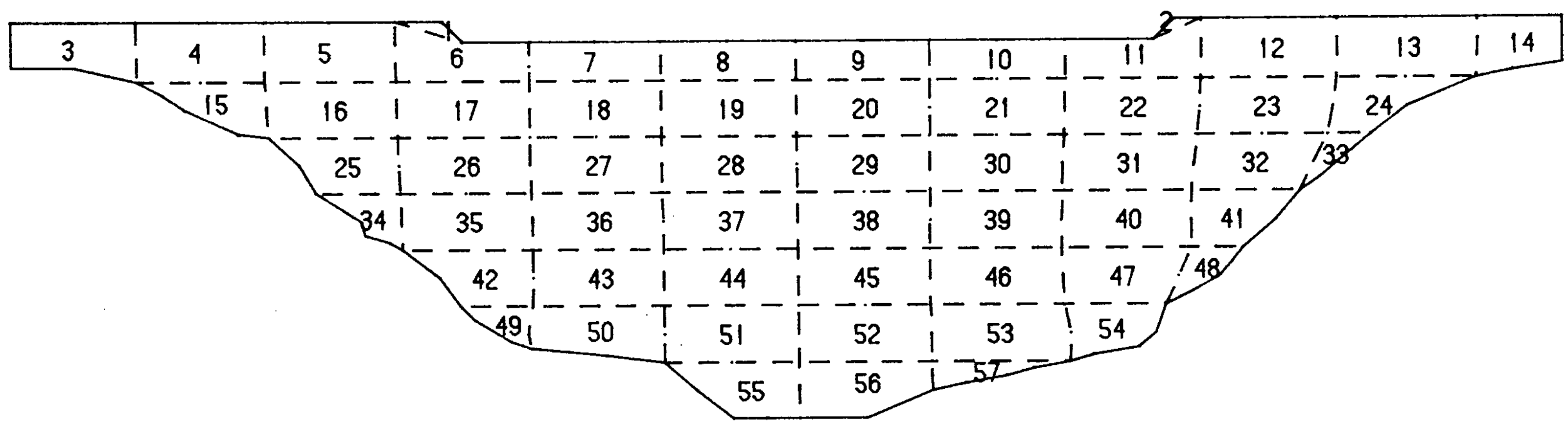


FIGURE 6-27 DEVELOPED VIEW OF DOWNSTREAM FACE, LOOKING UPSTREAM
ELEMENT NUMBERS ARE SHOWN

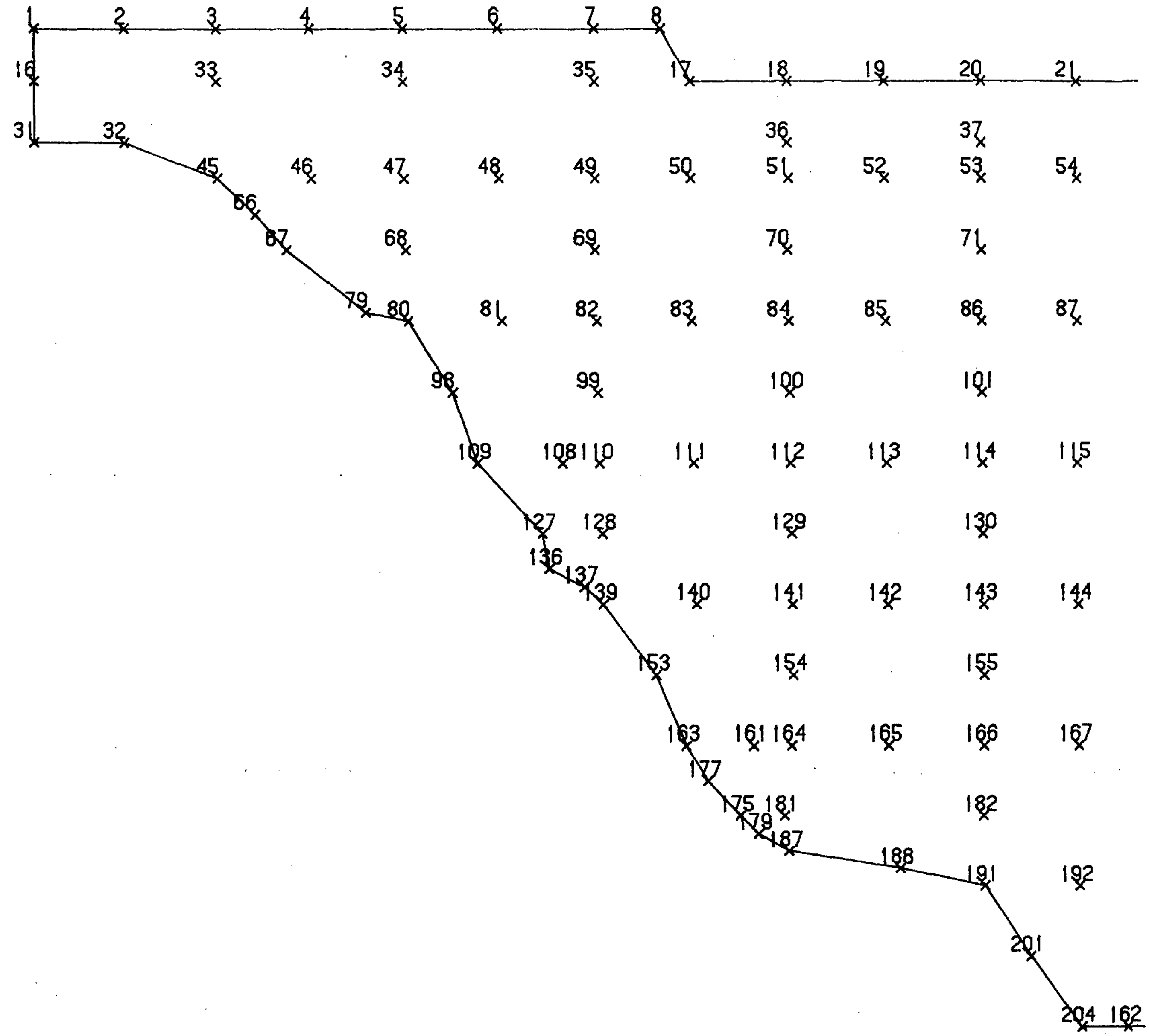


FIGURE 6-28 DEVELOPED VIEW O

NODE NU

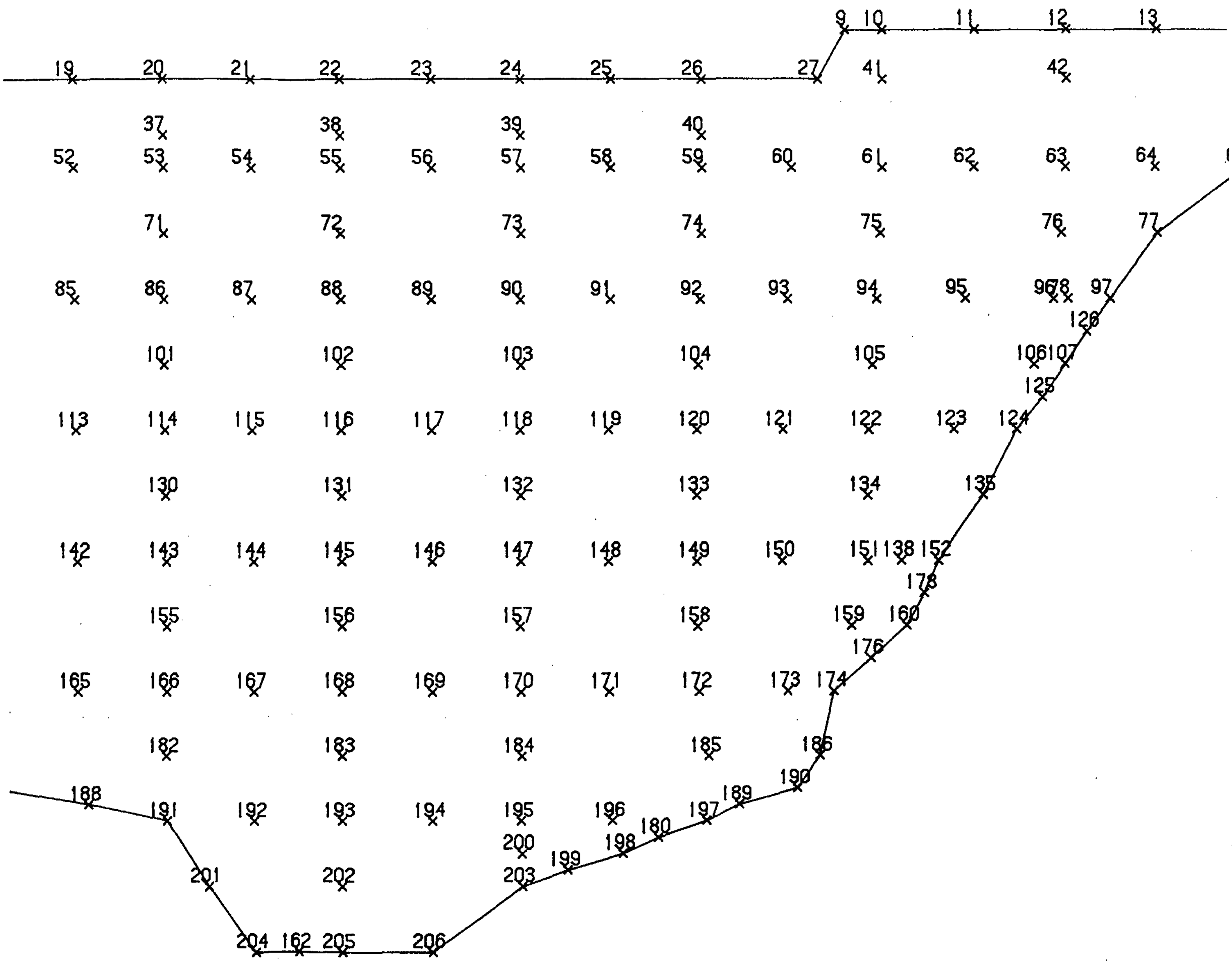
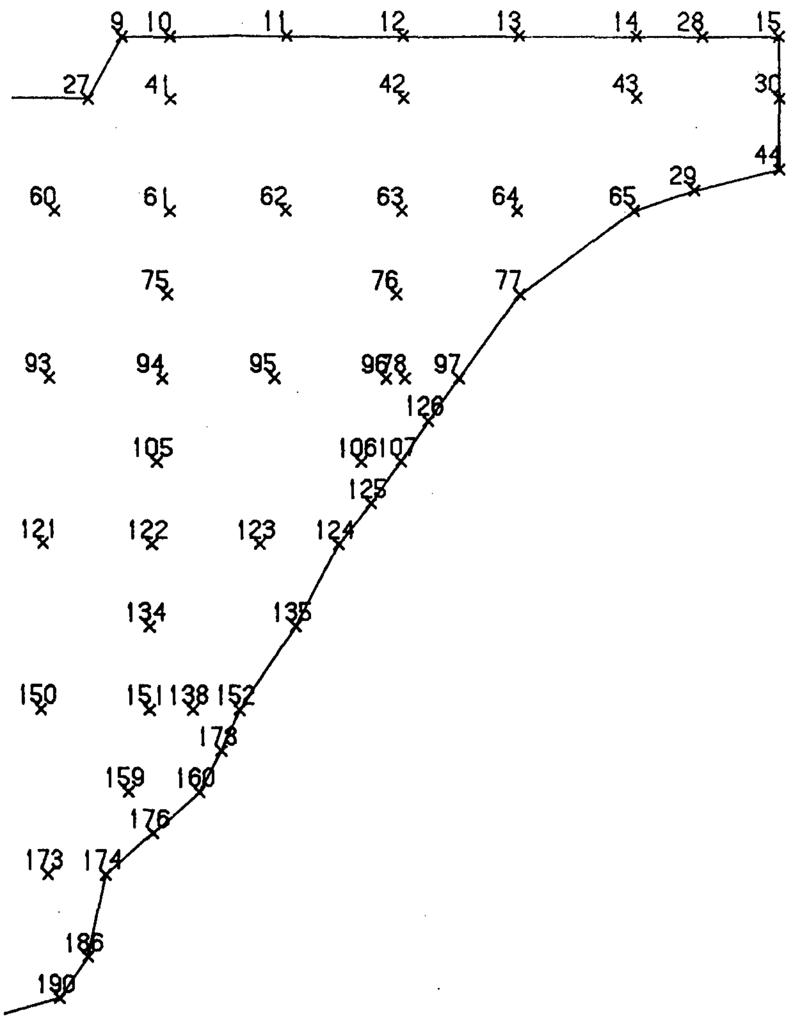


FIGURE 6-28 DEVELOPED VIEW OF DOWNSTREAM FACE, LOOKING UPSTREAM

NODE NUMBERS ARE SHOWN

2


9 10 11 12 13 14 28 15
27 41 42 43 30
44
60 61 62 63 64 65 29
75 76 77
93 94 95 96 78 97
126
105 106 107
125
121 122 123 124
134 135
150 151 138 152
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159 160
176
173 174
186
190

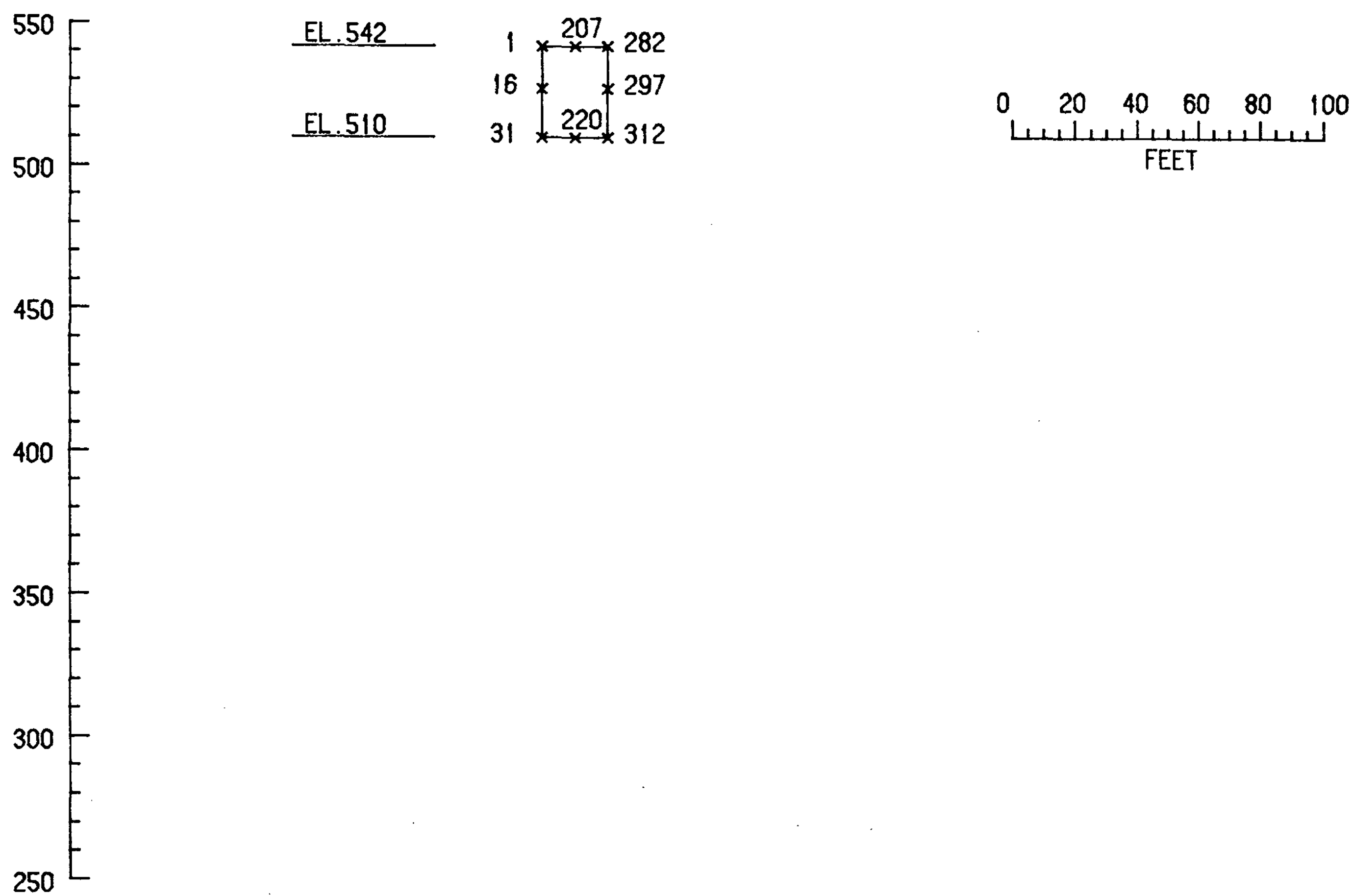


FIGURE 6-29 SECTION 1

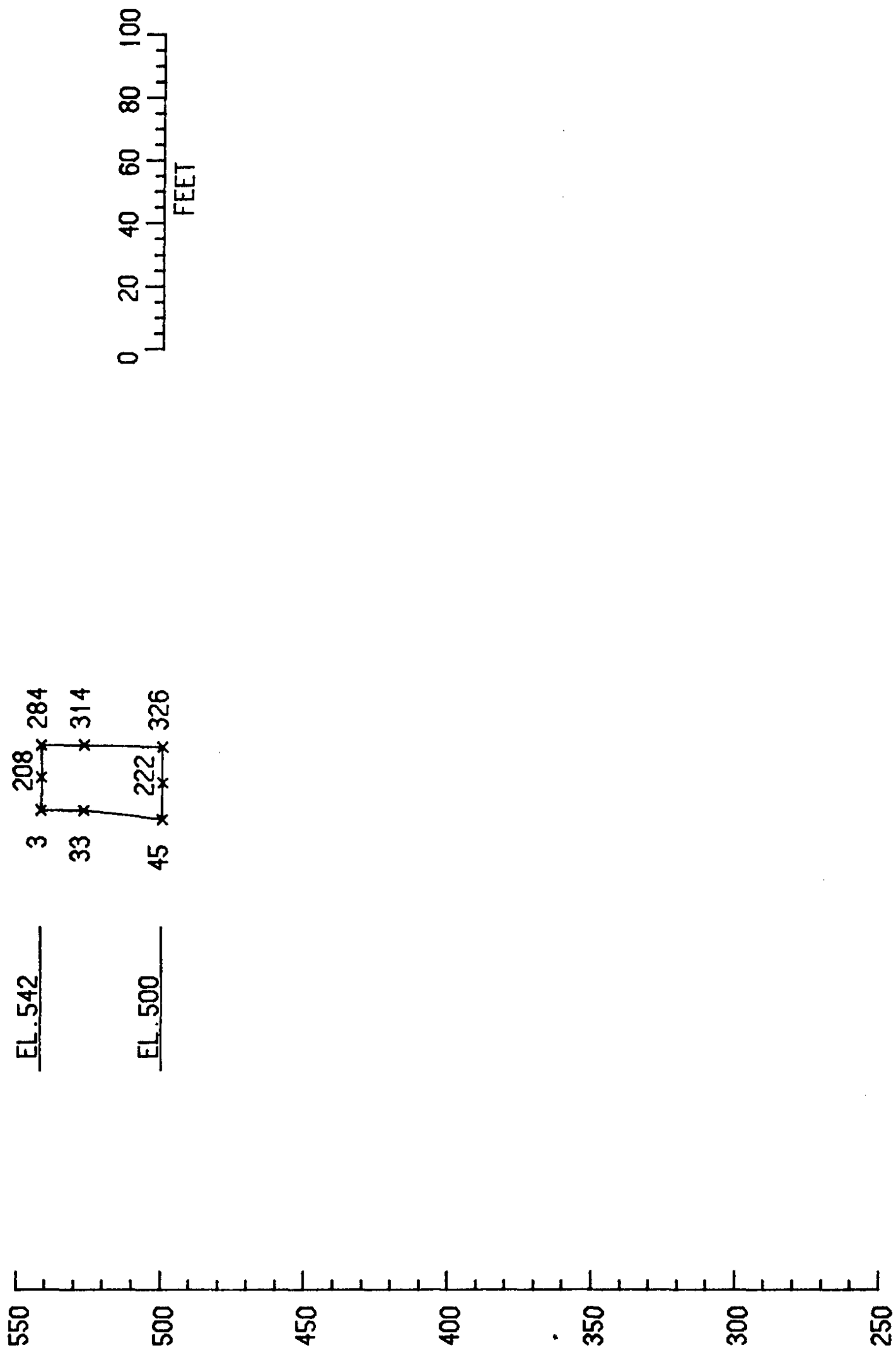


FIGURE 6-30 SECTION 2

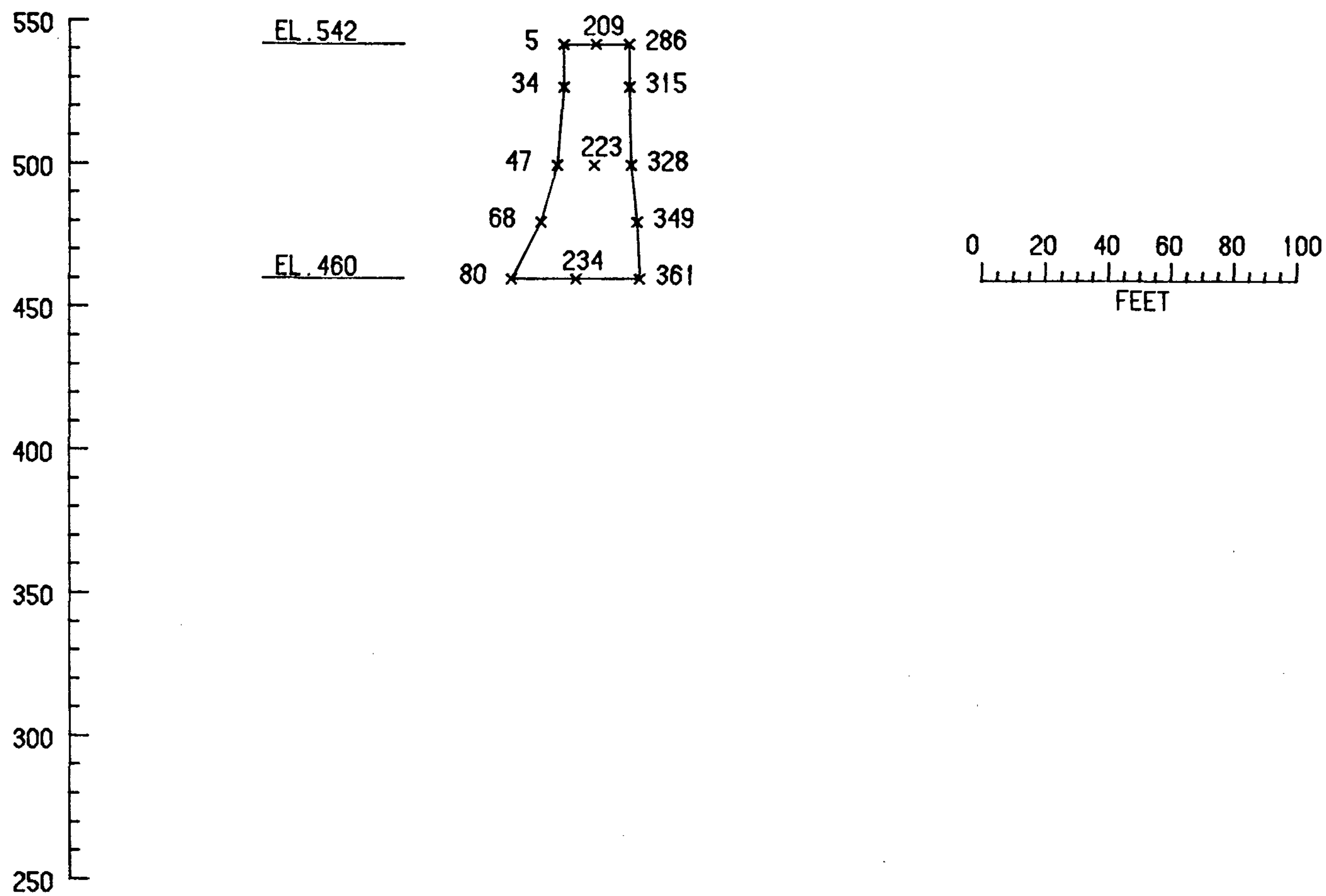


FIGURE 6-31 SECTION 3

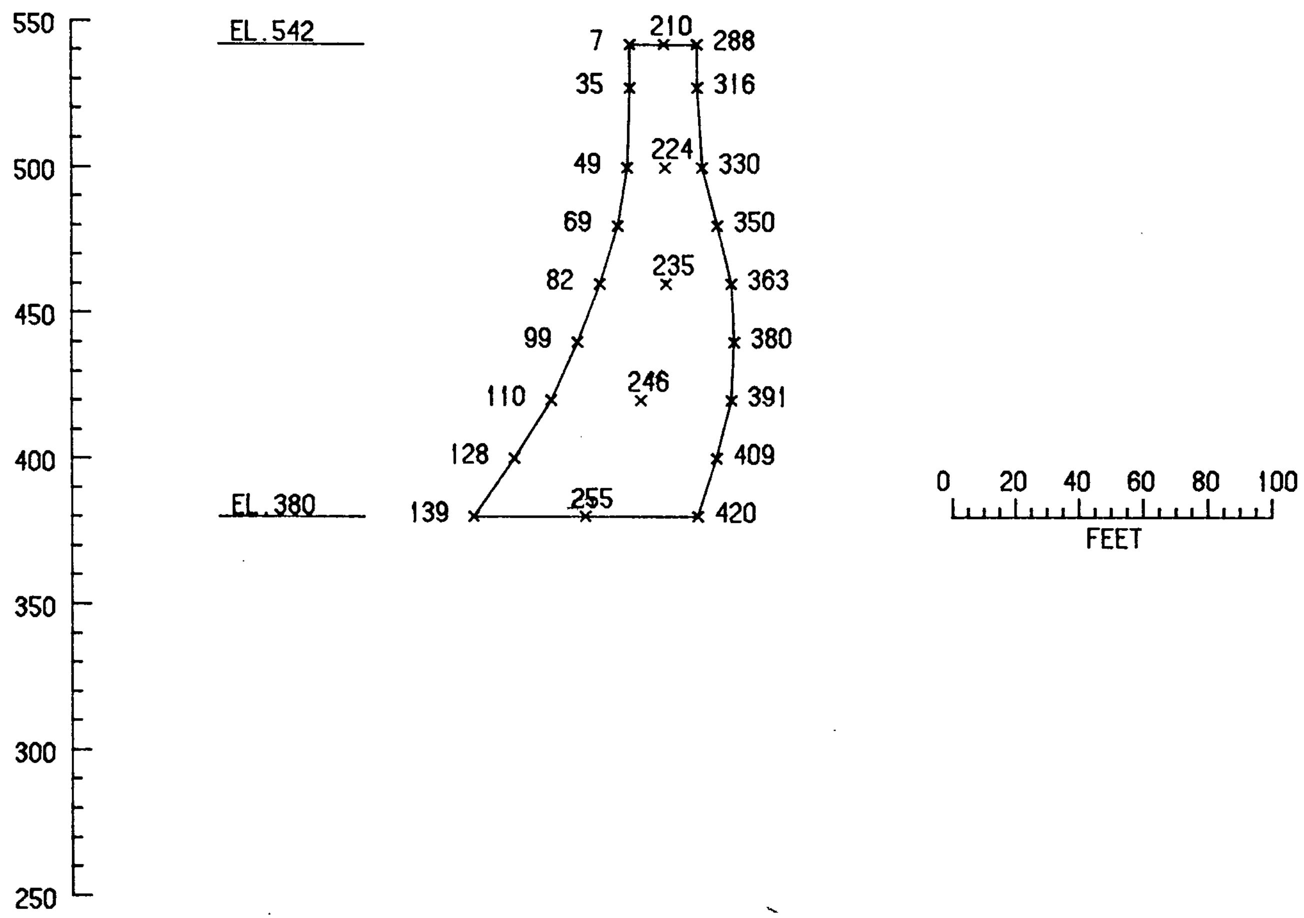


FIGURE 6-32 SECTION 4

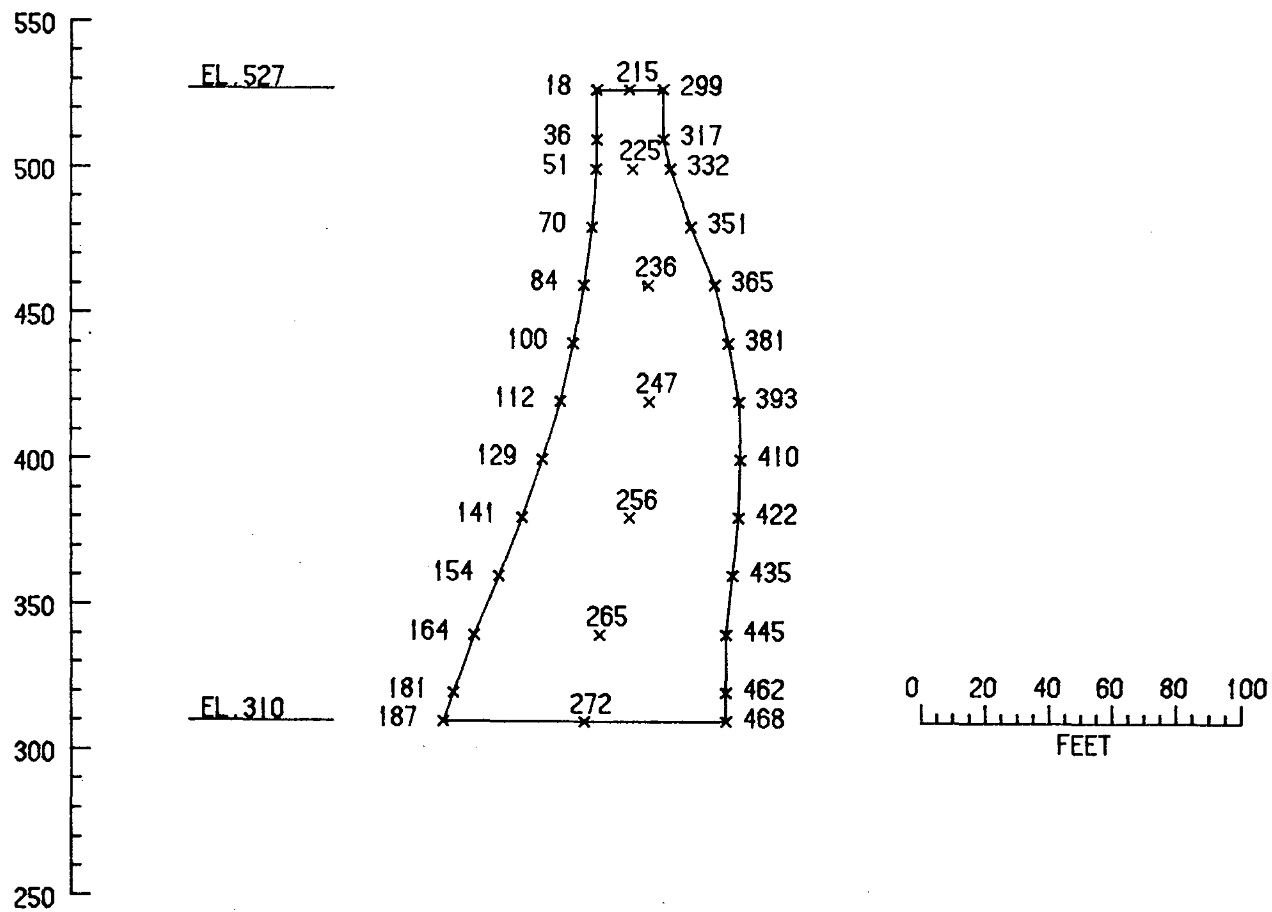


FIGURE 6-33 SECTION 5

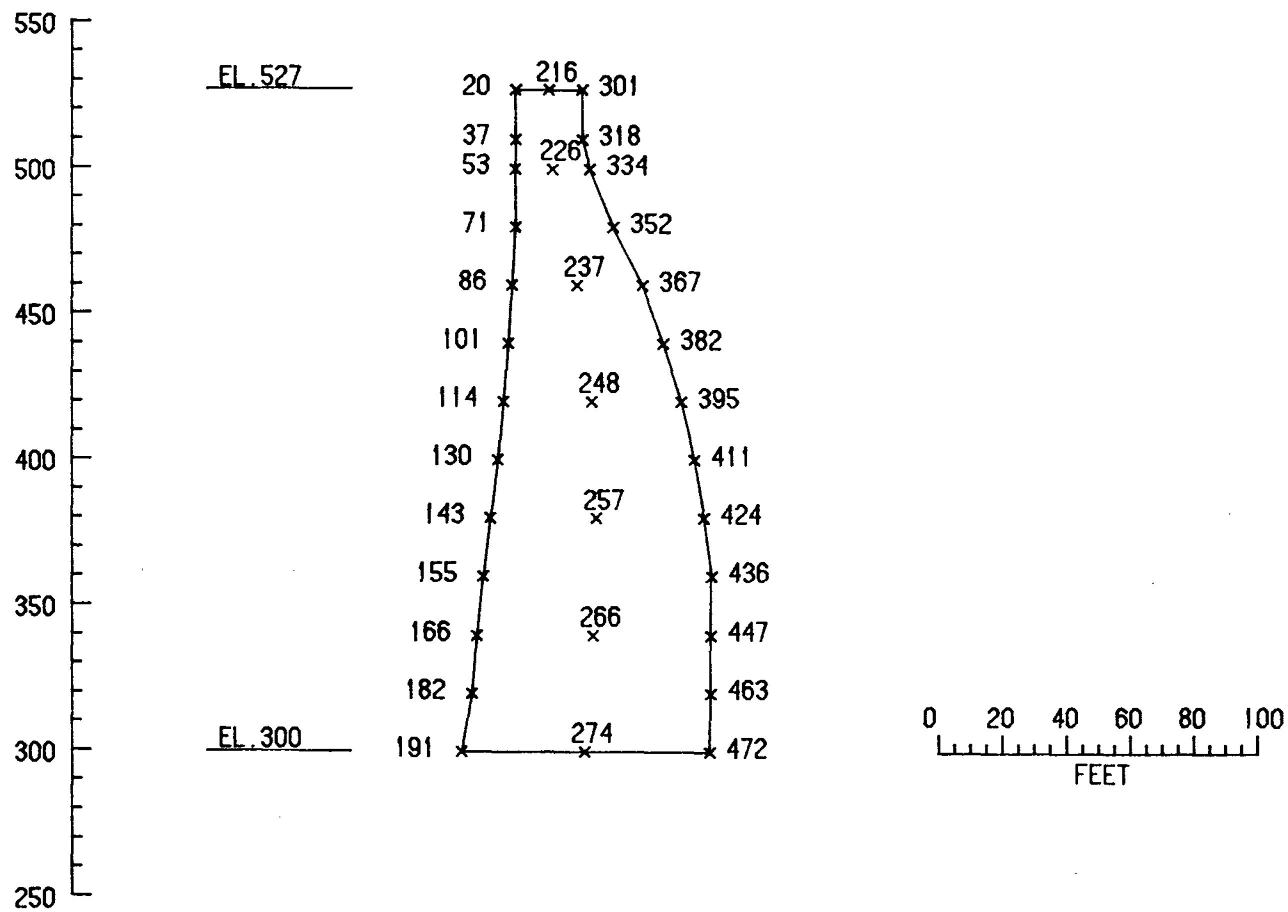


FIGURE 6-34 SECTION 6

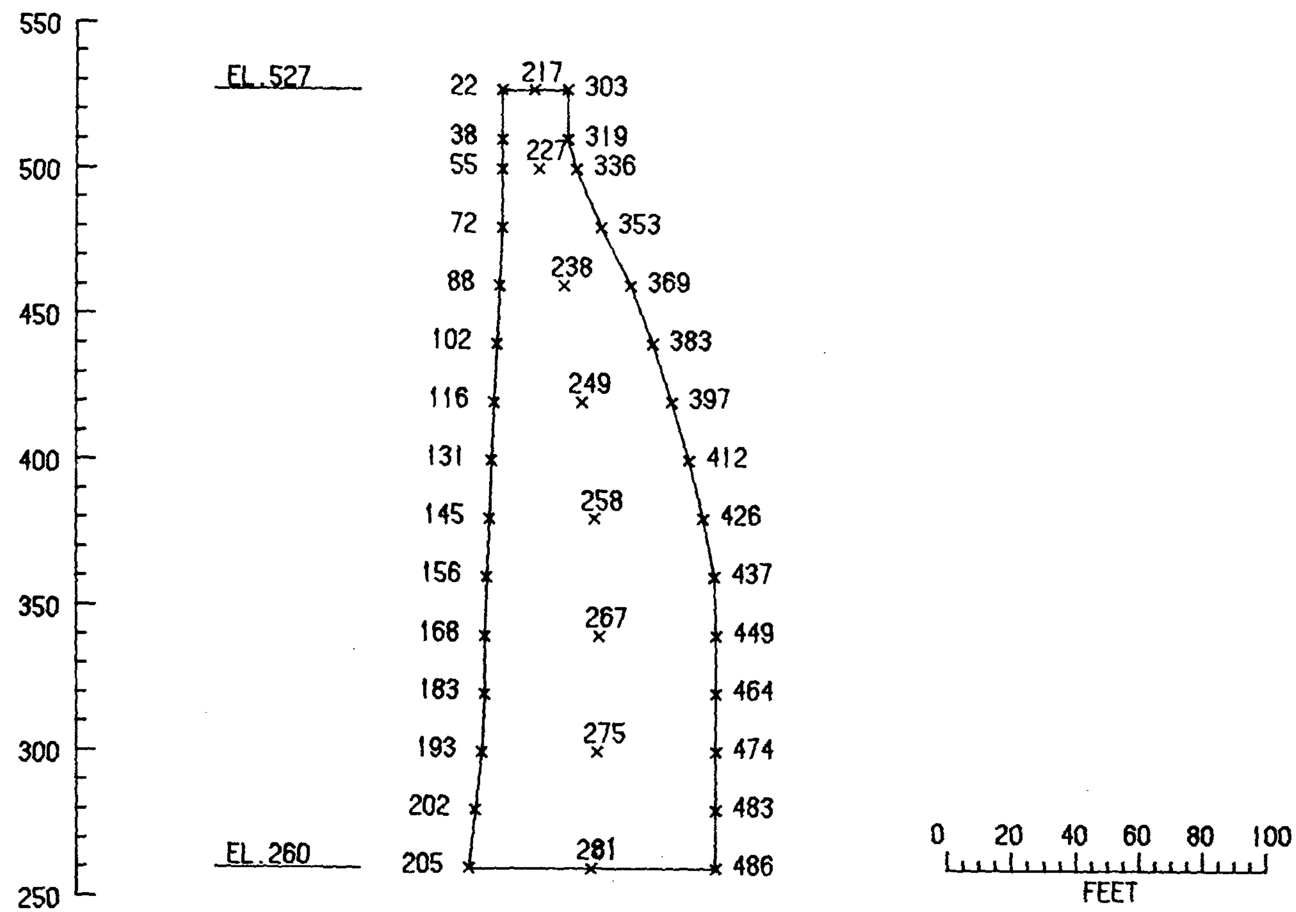


FIGURE 6-35 SECTION 7

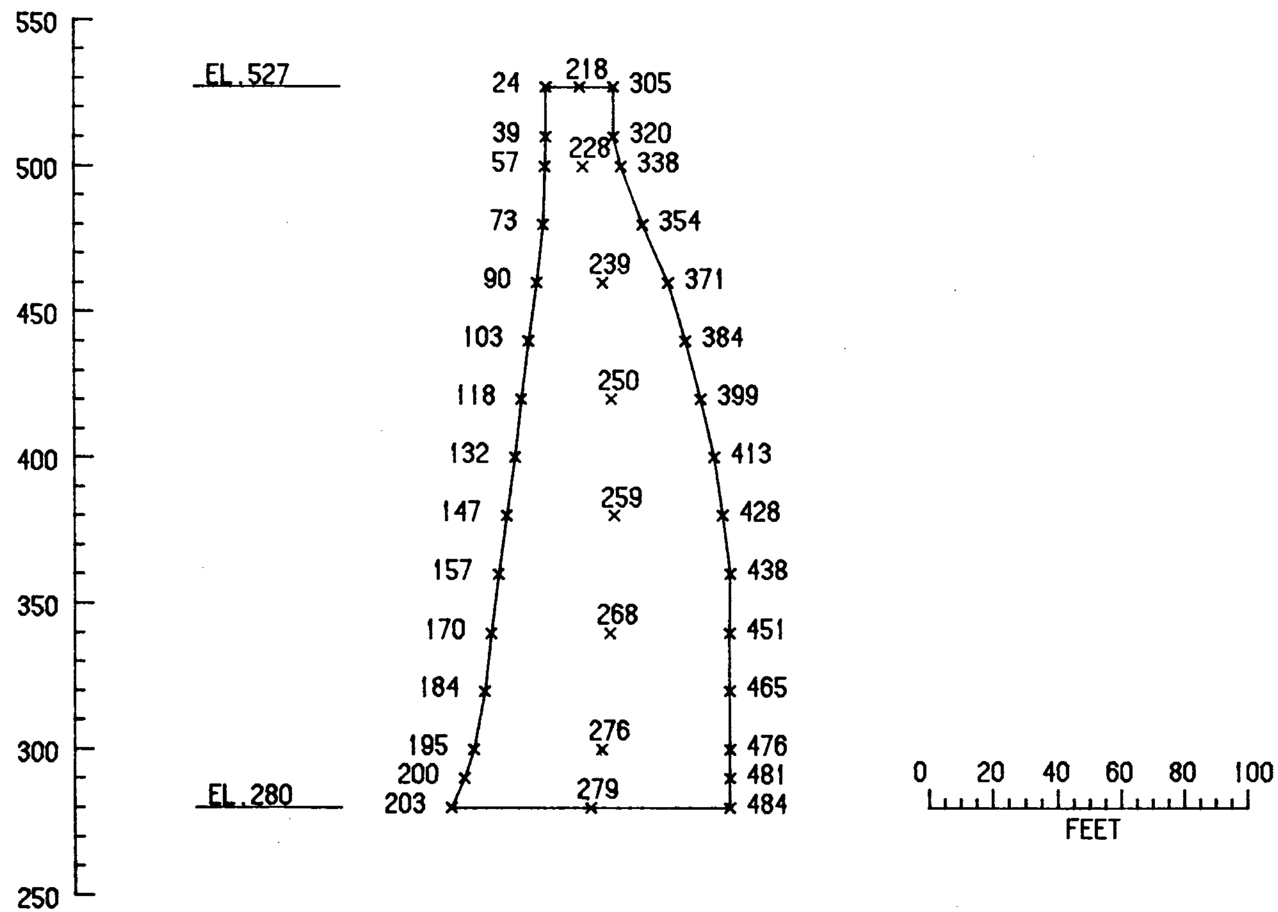


FIGURE 6-36 SECTION 8

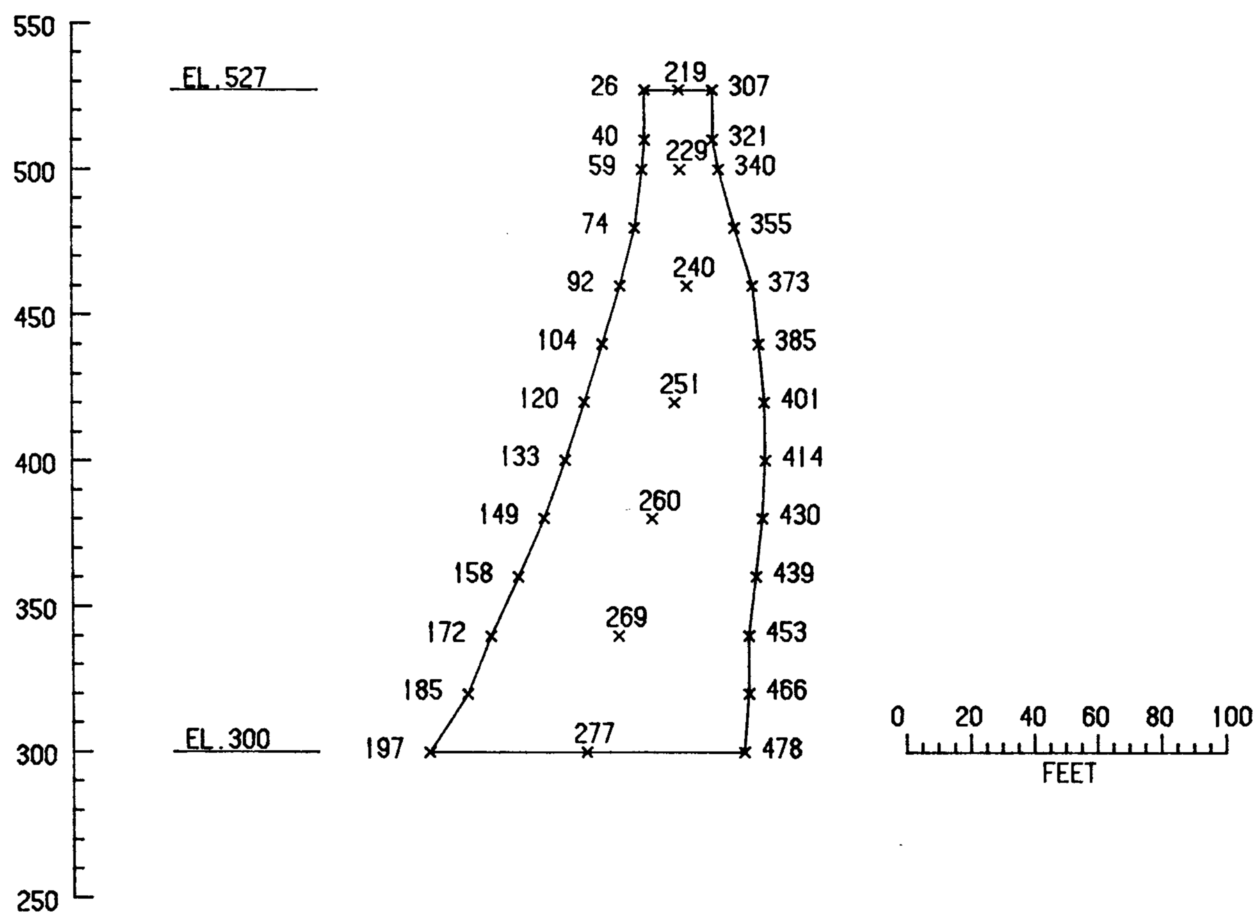


FIGURE 6-37 SECTION 9

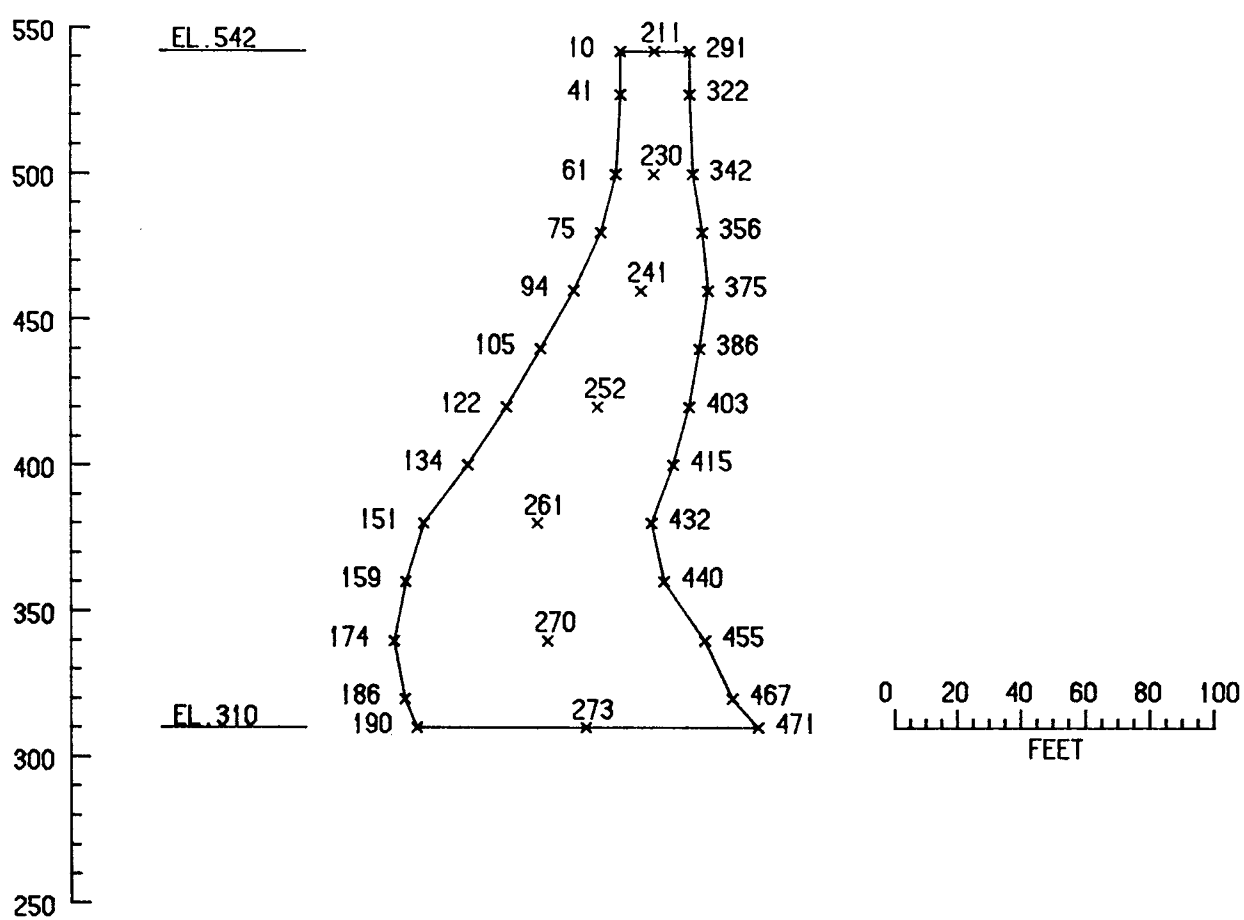


FIGURE 6-38 SECTION 10

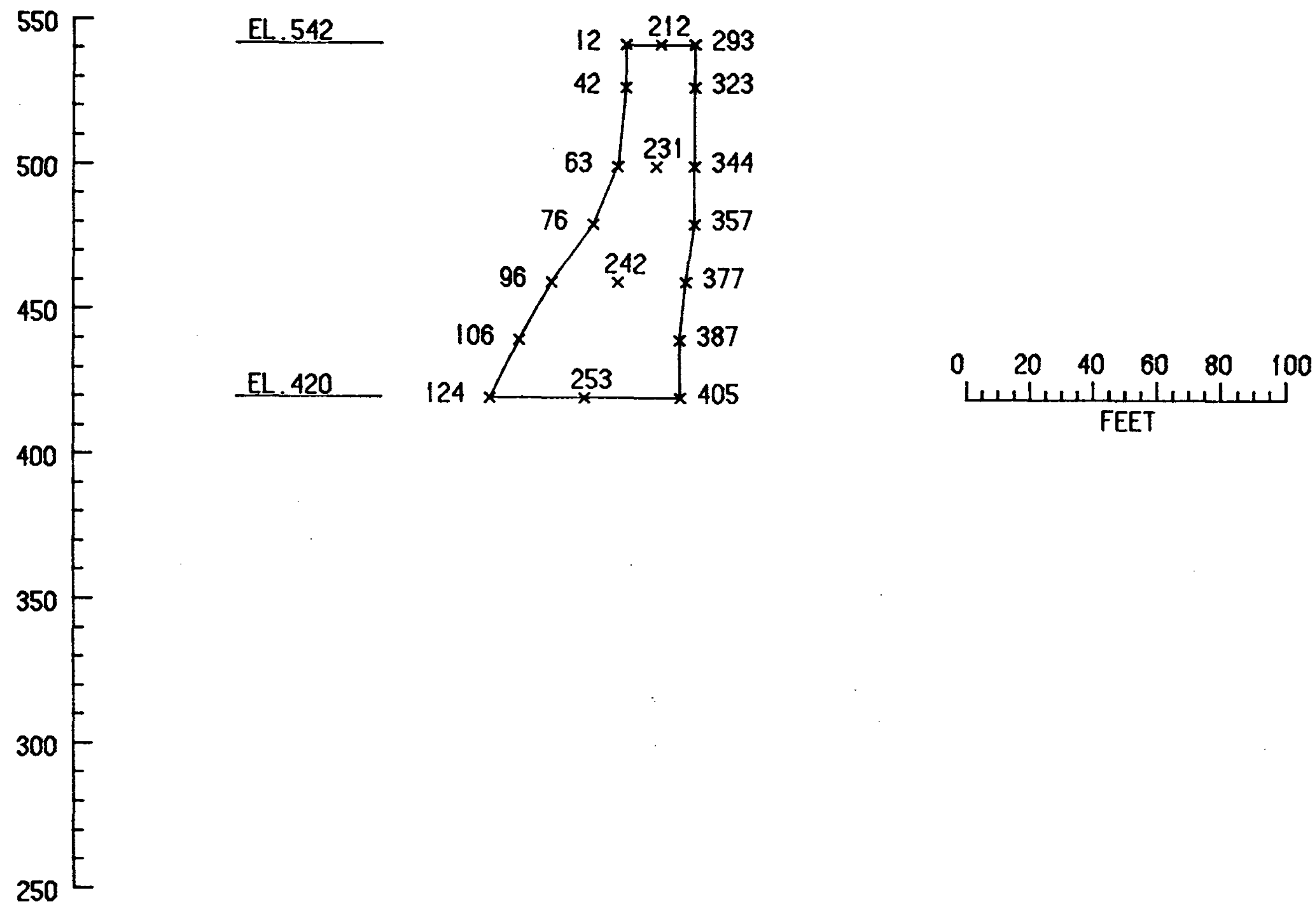


FIGURE 6-39 SECTION 11

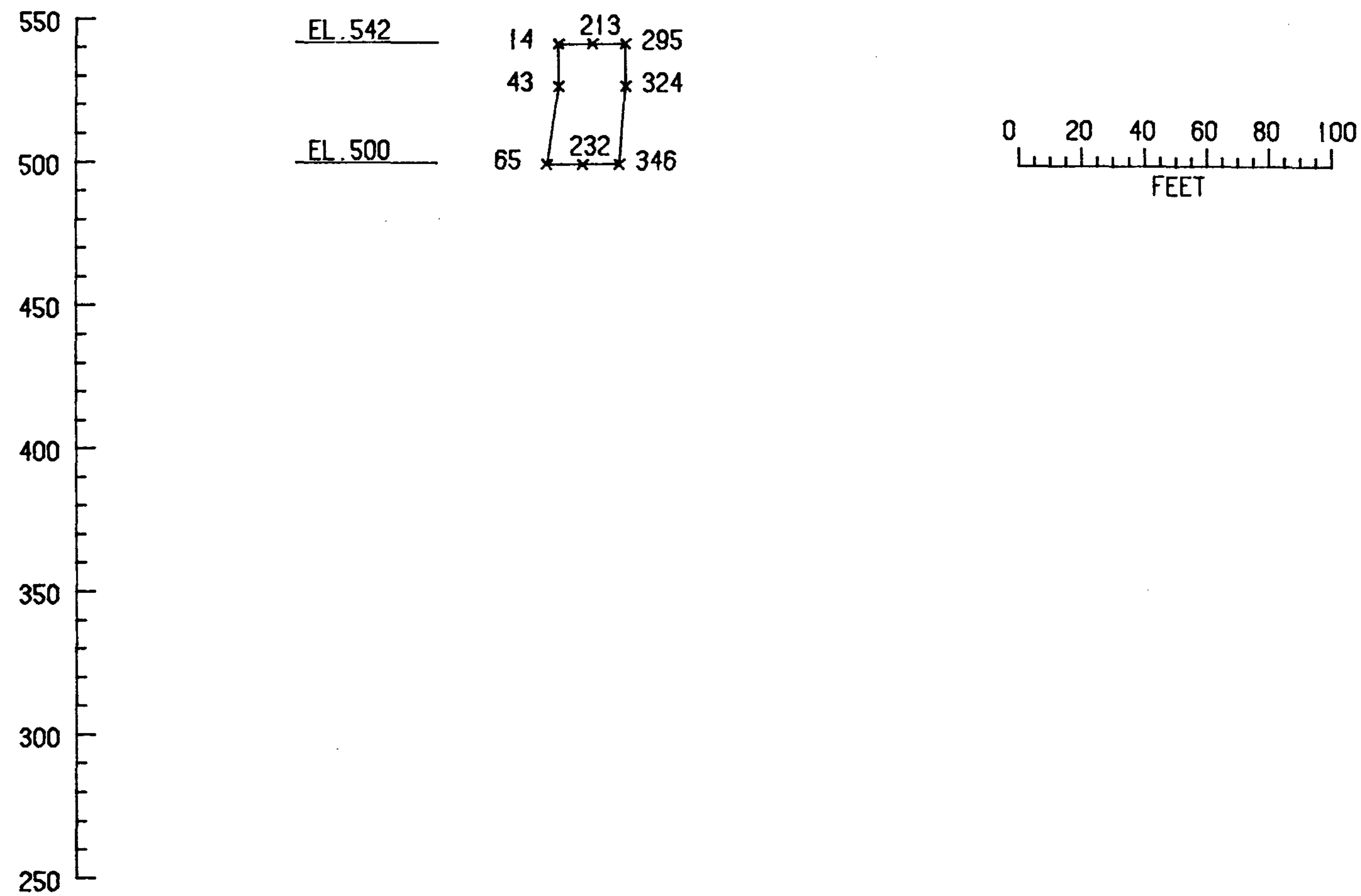


FIGURE 6-40 SECTION 12

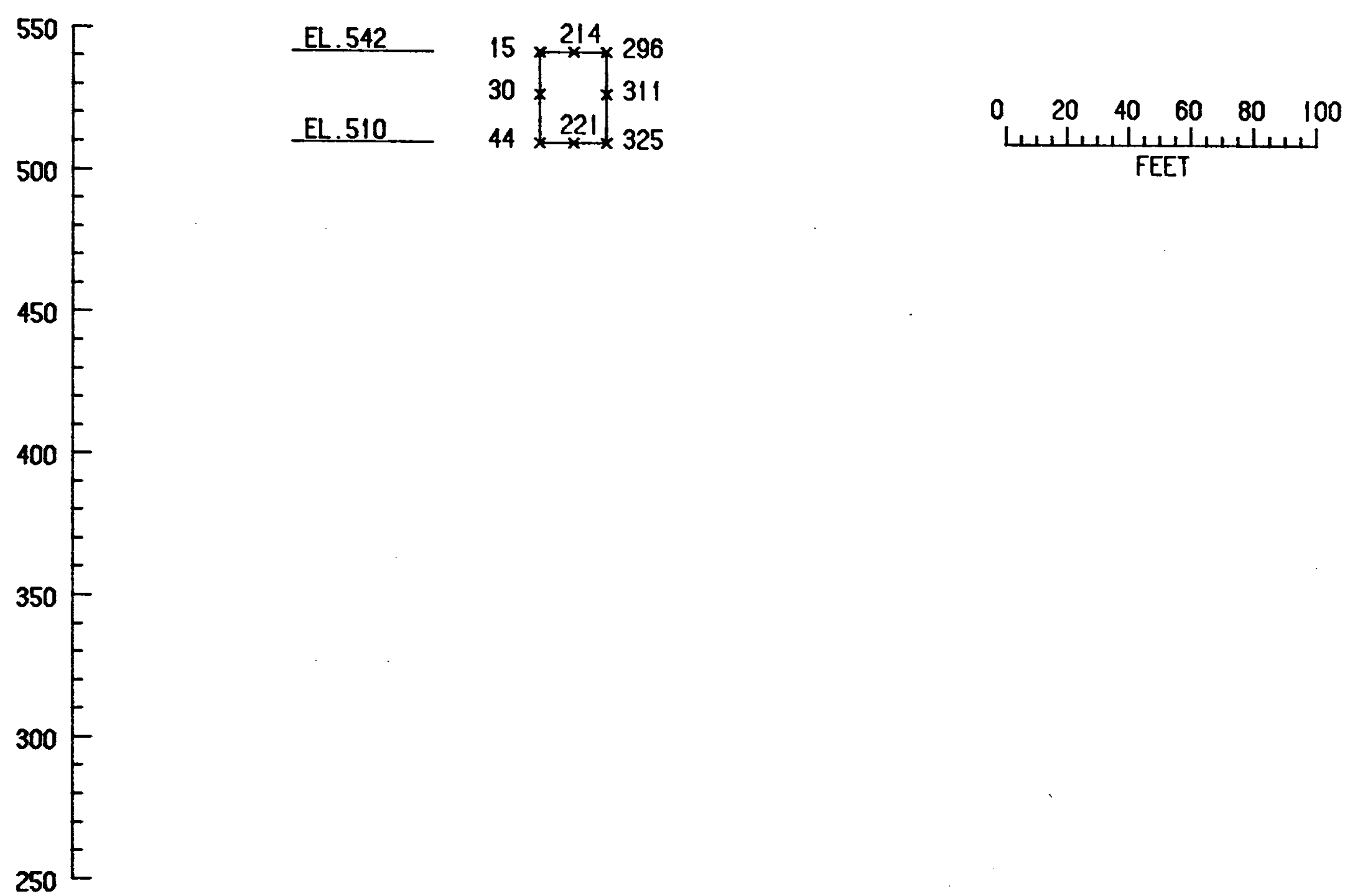
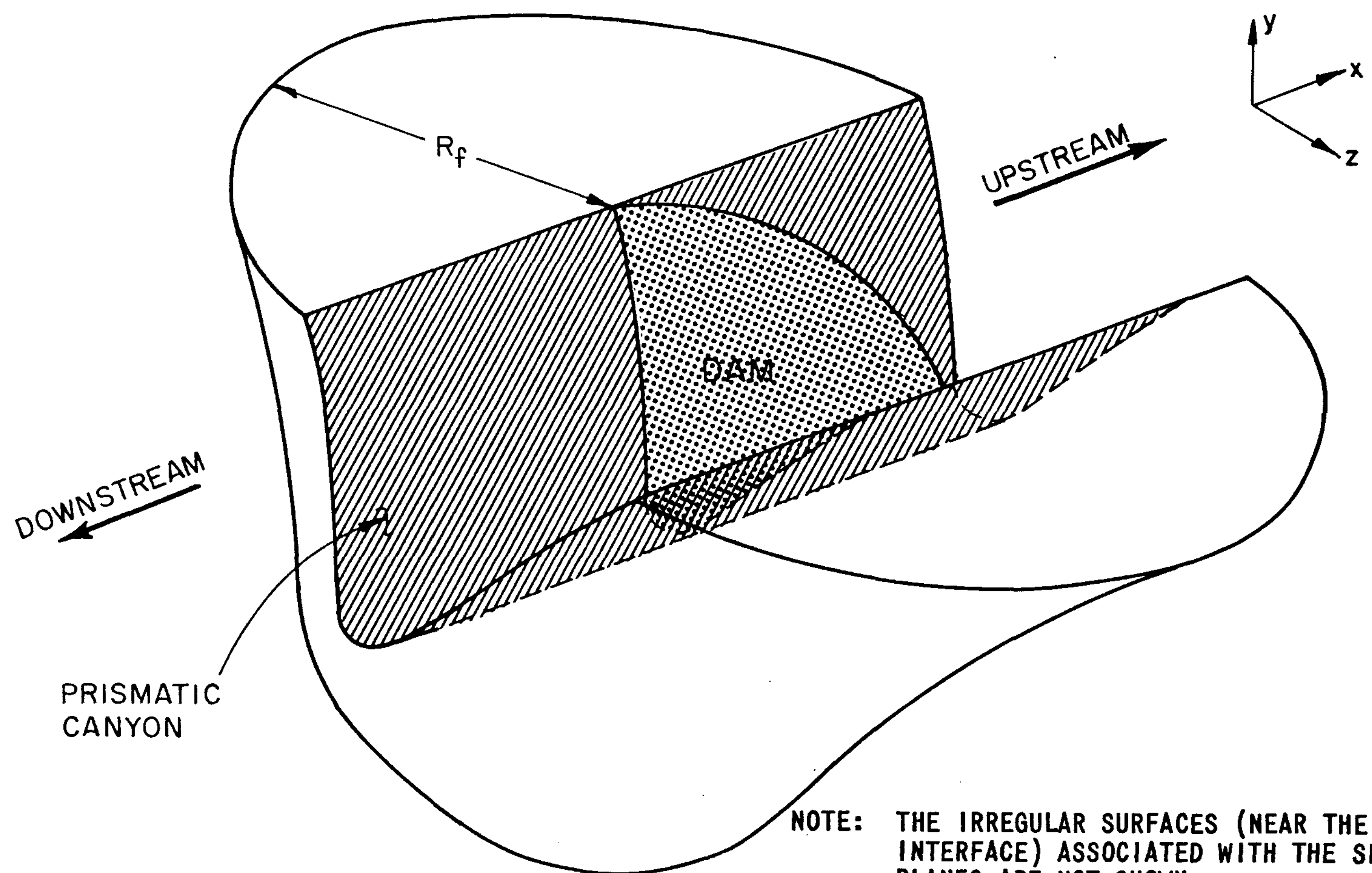


FIGURE 6-41 SECTION 13



NOTE: THE IRREGULAR SURFACES (NEAR THE DAM-FOUNDATION INTERFACE) ASSOCIATED WITH THE SEMICIRCULAR PLANES ARE NOT SHOWN.

FIGURE 6-42 IDEALIZED SHAPE OF FOUNDATION-ROCK REGION INCLUDED IN FINITE ELEMENT ANALYSIS

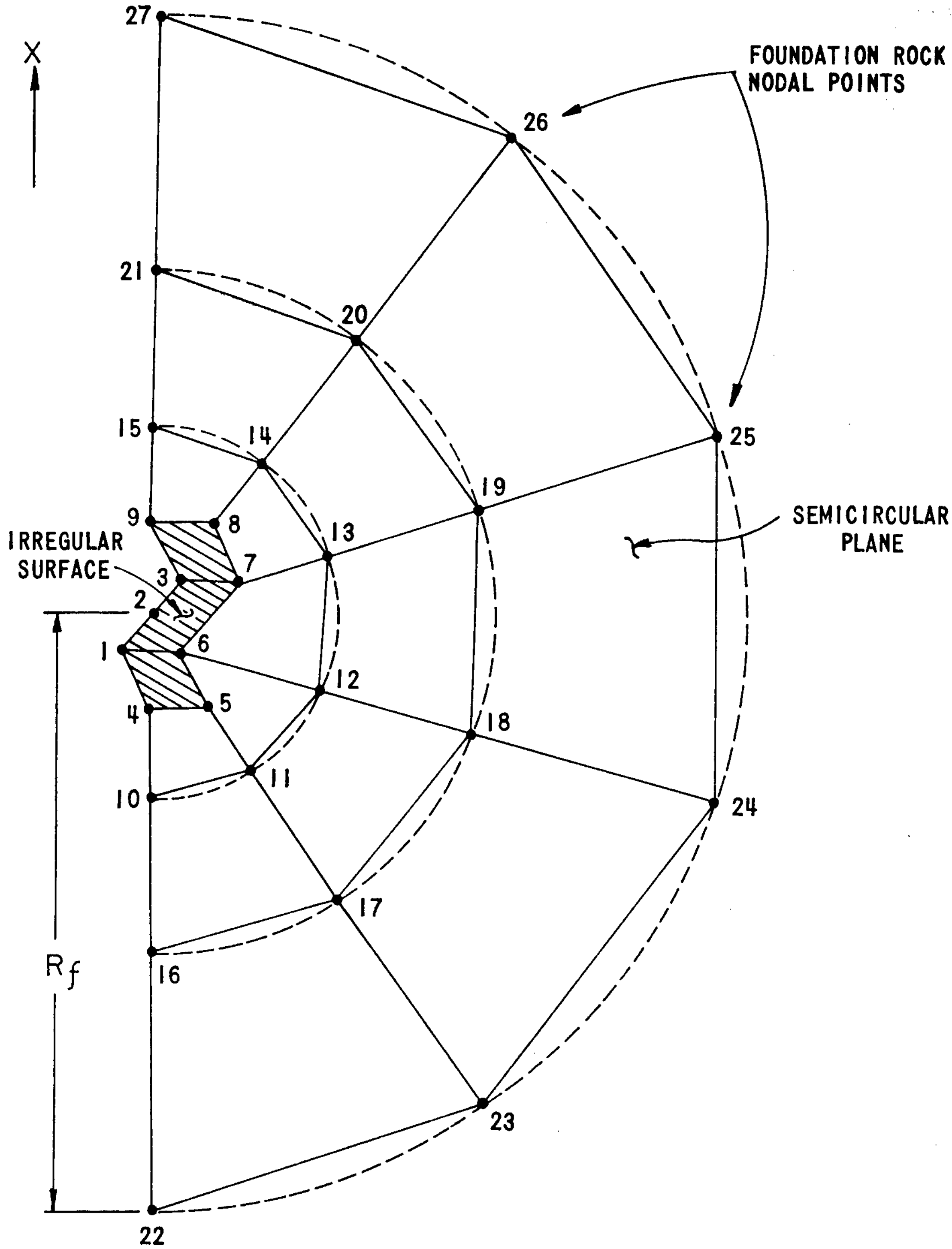
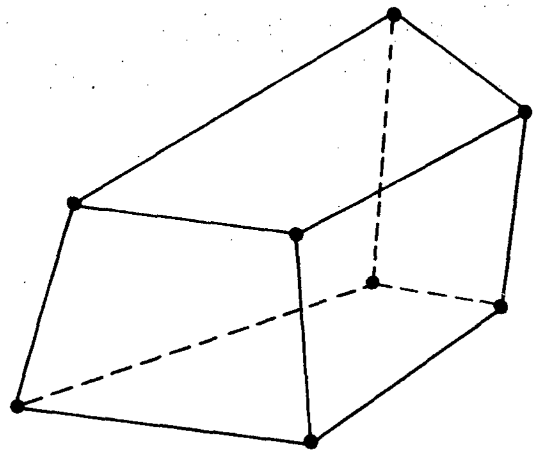
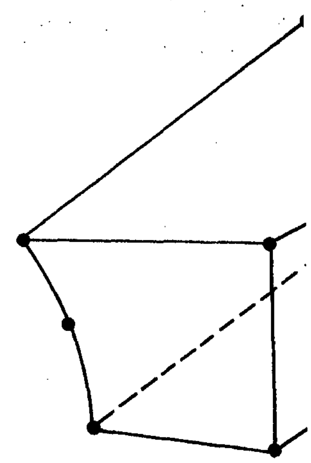


FIGURE 6-43 NODAL POINT ARRANGEMENT ON AN INCLINED SEMICIRCULAR PLANE AND IRREGULAR SURFACE CONNECTING THE PLANE TO THE DAM-FOUNDATION INTERFACE



TYPICAL 8-NODE SOLID FOUNDATION ELEMENT

NOTE: THIS TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL FACES AND IS USED IN BOTH EACD AND GTSTRUDL ANALYSES.



TYPICAL 9-NODE SOLID FOUNDATION ELEMEN

NOTE: THIS TYPE OF SOLID ELEMENT HAS
THE EDGE WITH THREE NODES MAY B
SHAPE. THE 9-NODE ELEMENT IS U
IN THE CASE OF GTSTRUDL ANALYSI
OMITTED AND THE ELEMENT BECOMES

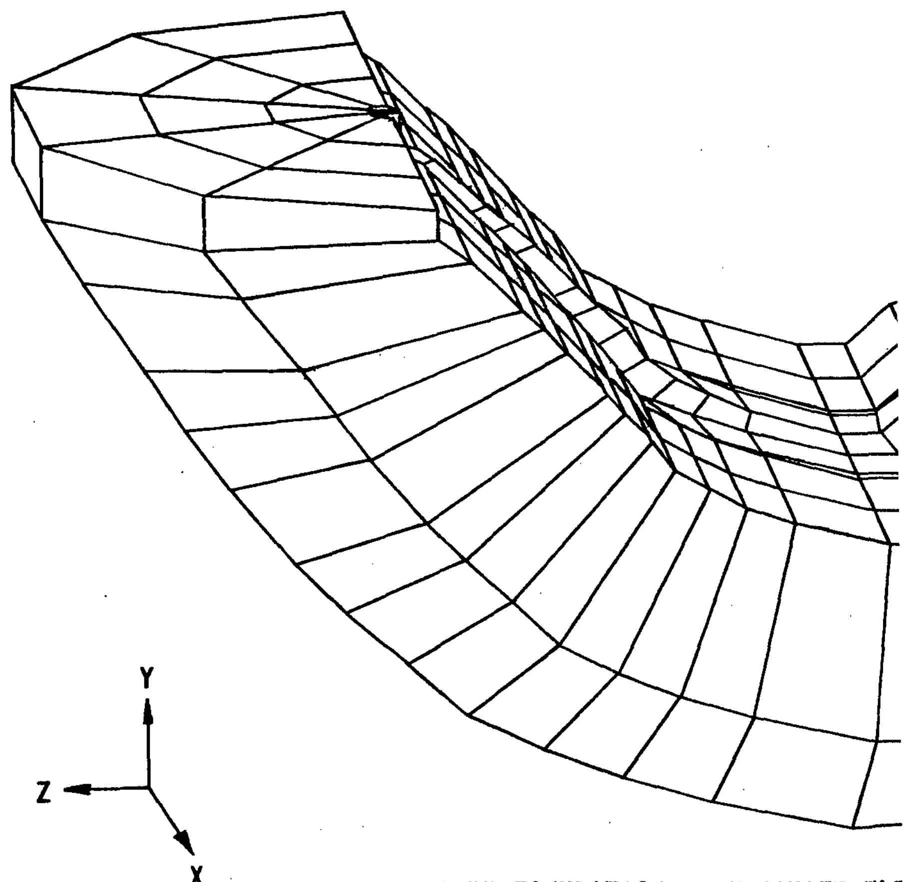
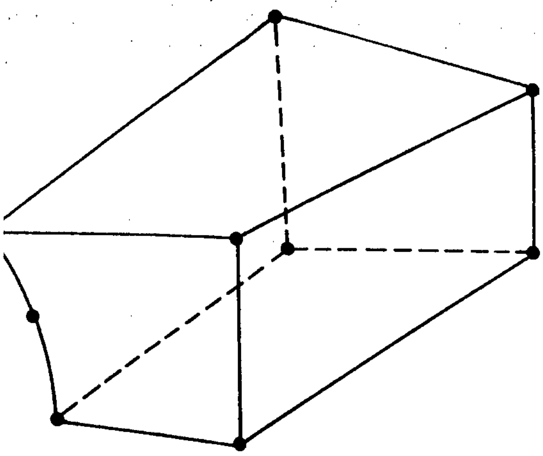
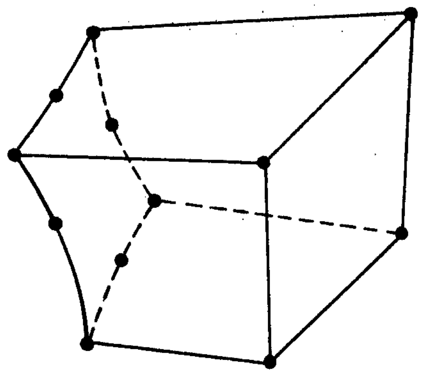


FIGURE 6-44 FOUNDATION-ROCK FINITE ELE



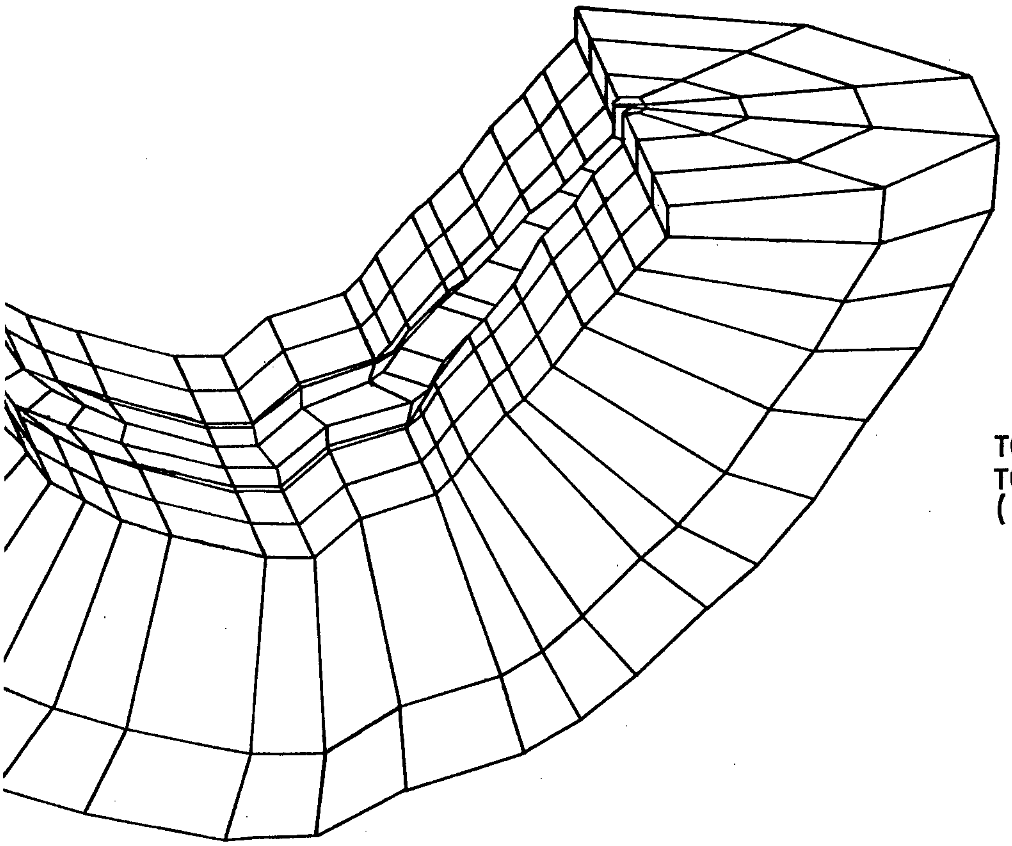
FOUNDATION ELEMENT

;OLID ELEMENT HAS SIX QUADRILATERAL FACES.
THREE NODES MAY BE CURVED TO A PARABOLIC
-NODE ELEMENT IS USED IN EACD ANALYSIS ONLY.
: GTSTRUDL ANALYSIS, THE NON-CORNER NODE IS
IE ELEMENT BECOMES 8-NODE SOLID ELEMENT.



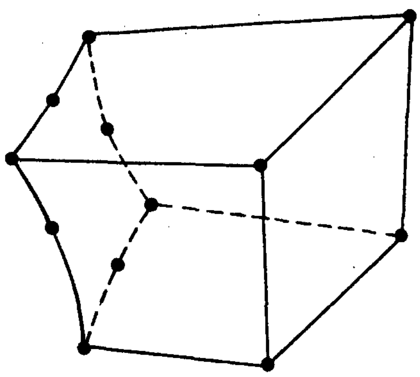
TYPICAL 12-NODE SOLID FOUNDATION ELEMENT

NOTE: THIS TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL
THE EDGES WITH THREE NODES MAY BE CURVED TO A PA
SHAPE. THE 12-NODE ELEMENT IS USED IN EACD ANAL
IN THE CASE OF GTSTRUDL ANALYSIS, THE NON-CORNER
ARE OMITTED AND THE ELEMENT BECOMES 8-NODE SOLID



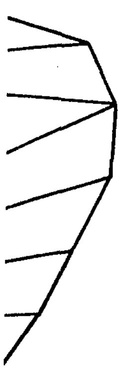
TOTAL NUMBER OF FOUNDATION ELEMENTS =
TOTAL NUMBER OF FOUNDATION NODES
(INCLUDING NON-CORNER NODES) = 781

ON-ROCK FINITE ELEMENT MODEL, LOOKING DOWNSTREAM



NODE SOLID FOUNDATION ELEMENT

TYPE OF SOLID ELEMENT HAS SIX QUADRILATERAL FACES.
EDGES WITH THREE NODES MAY BE CURVED TO A PARABOLIC
E. THE 12-NODE ELEMENT IS USED IN EACD ANALYSIS ONLY.
HE CASE OF GTSTRUDL ANALYSIS, THE NON-CORNER NODES
OMITTED AND THE ELEMENT BECOMES 8-NODE SOLID ELEMENT.



TOTAL NUMBER OF FOUNDATION ELEMENTS = 468
TOTAL NUMBER OF FOUNDATION NODES
(INCLUDING NON-CORNER NODES) = 781

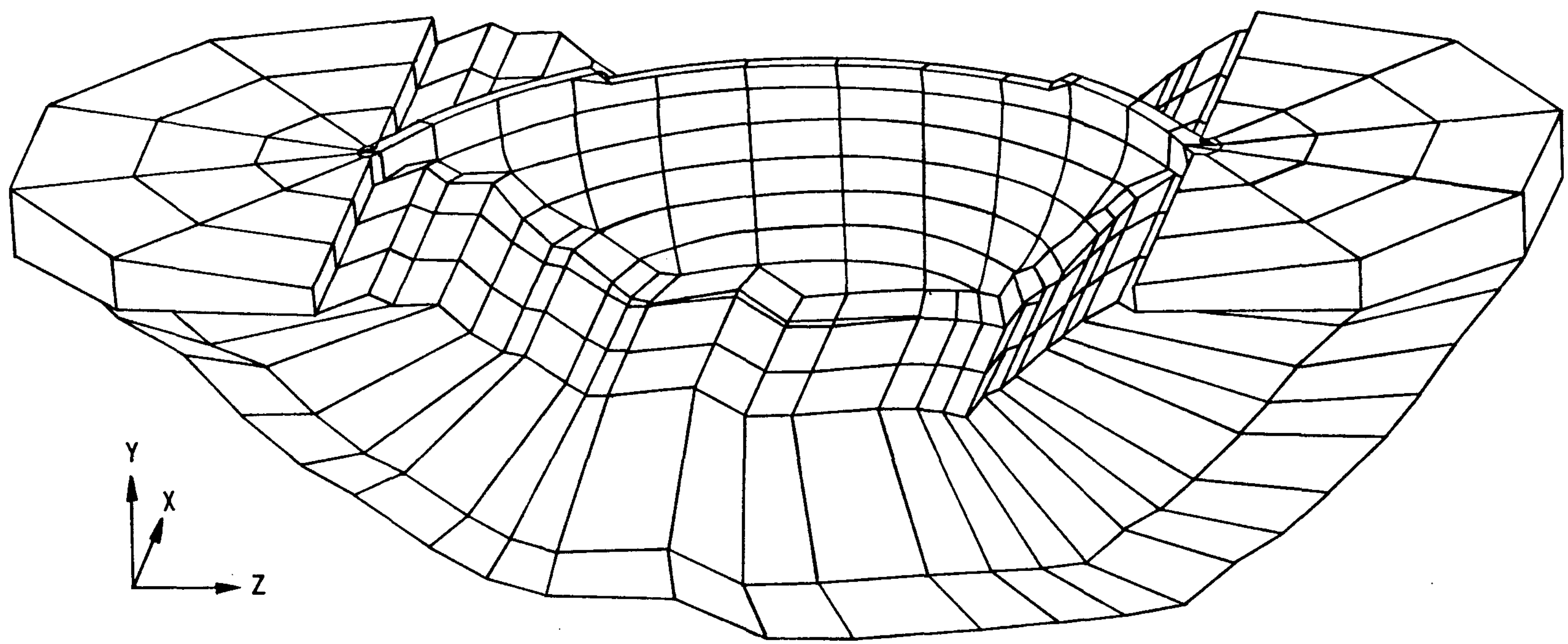


FIGURE 6-45 FINITE ELEMENT MESH: DAM AND FOUNDATION, LOOKING UPSTREAM

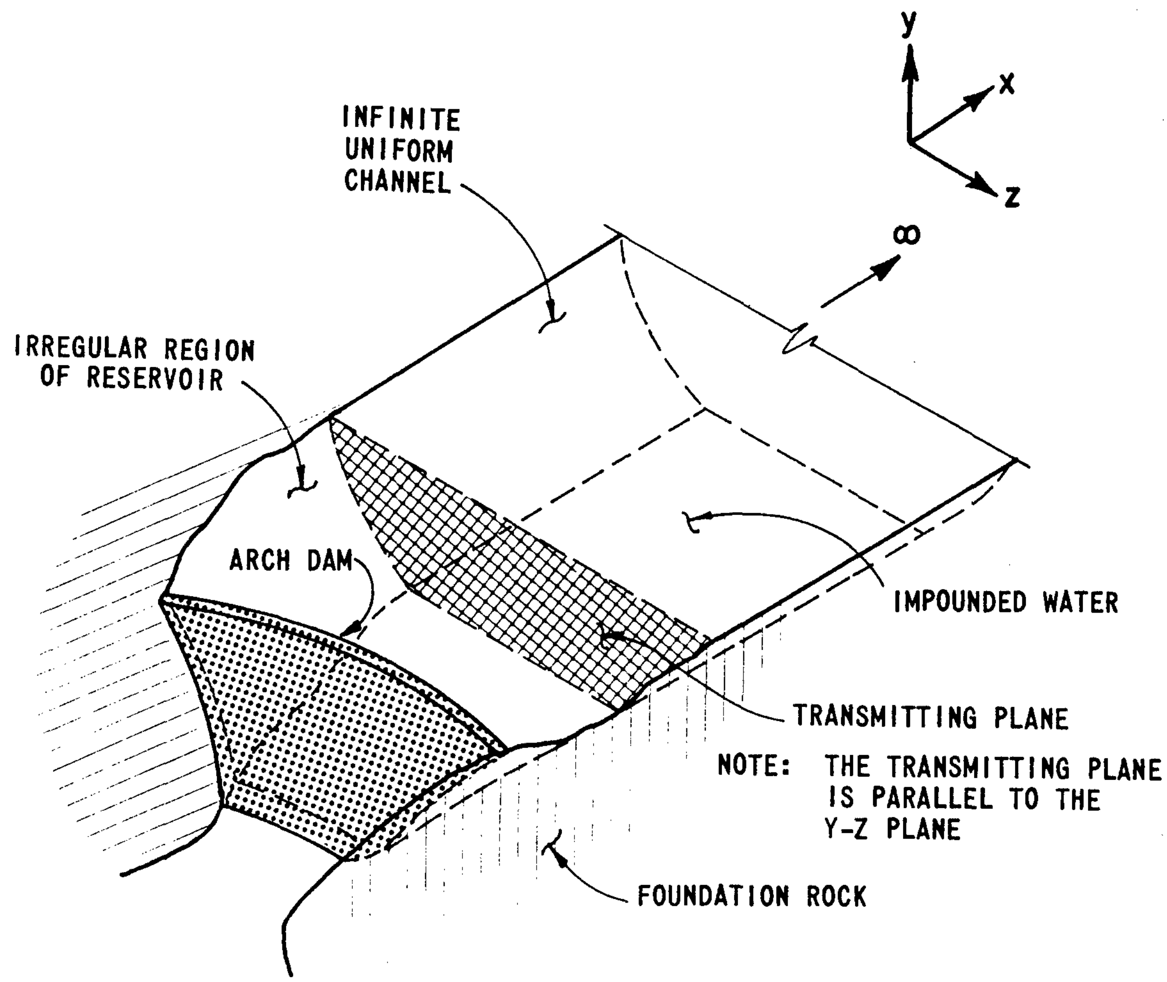
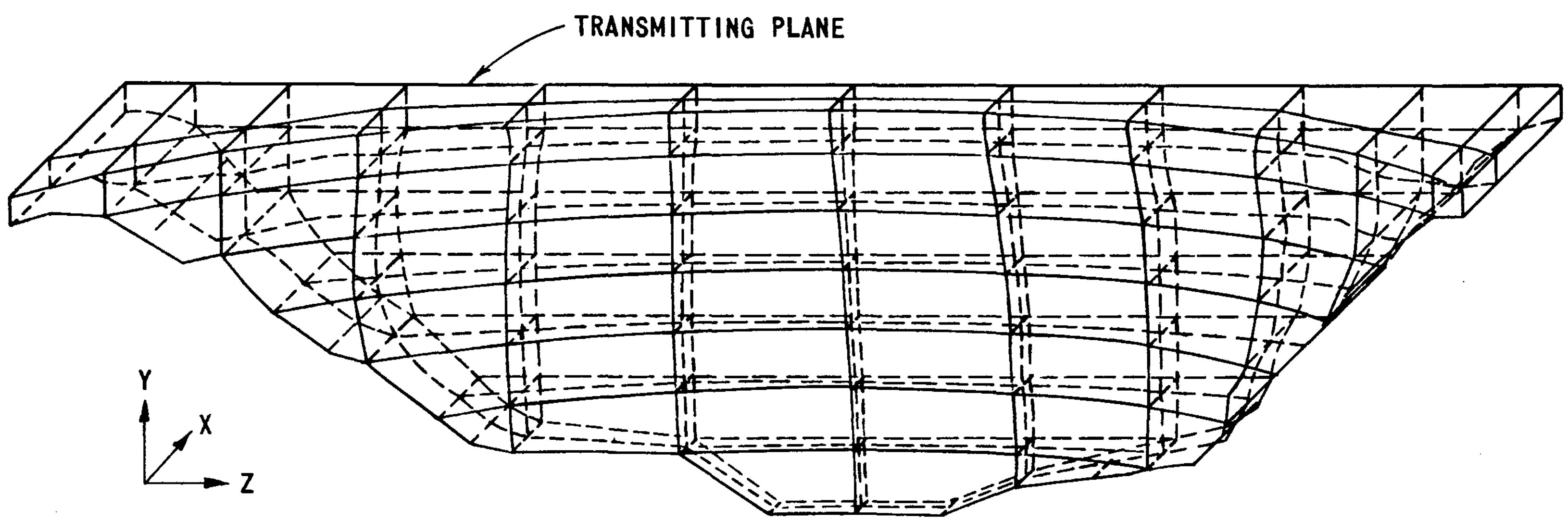
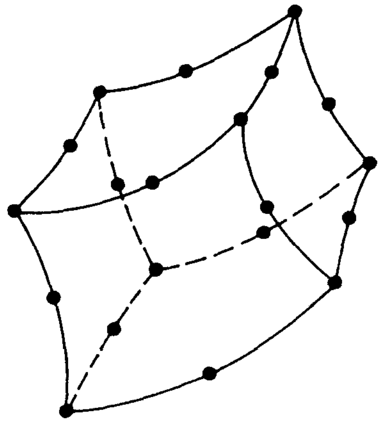


FIGURE 6-46 ILLUSTRATIVE SKETCH OF ARCH DAM-WATER-FOUNDATION SYSTEM



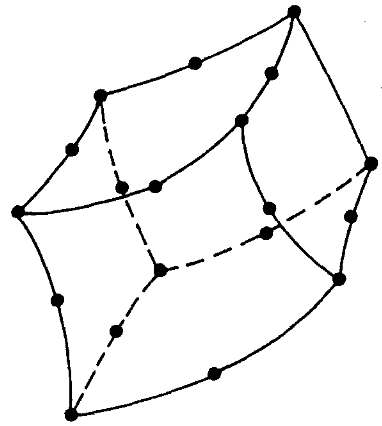
TOTAL NUMBER OF 3-DIMENSIONAL MESH I
ELEMENTS = 110
TOTAL NUMBER OF FLUID NODAL POINTS = 748

FIGURE 6-47 RESERVOIR FINITE ELEMENT MODEL, LOOKING UPSTREAM



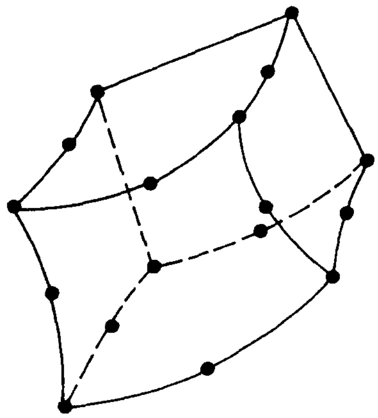
TYPICAL 3-D 20-NODE ELEMENT

MESH 1



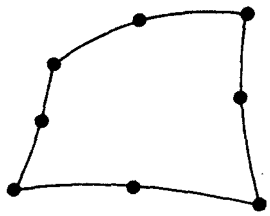
TYPICAL 3-D 19-NODE ELEMENT

MESH 1



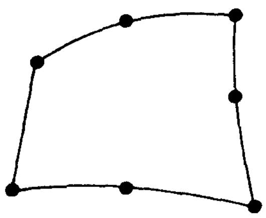
TYPICAL 3-D 17-NODE ELEMENT

MESH 1



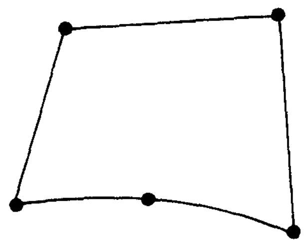
TYPICAL 2-D 8-NODE ELEMENT

MESH 2, MESH 3, MESH 4



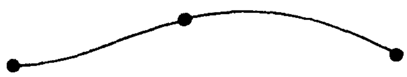
TYPICAL 2-D 7-NODE ELEMENT

MESH 3, MESH 4



TYPICAL 2-D 5-NODE ELEMENT

MESH 3



TYPICAL 1-D 3-NODE ELEMENT

MESH 5

FIGURE 6-48 TYPES OF FLUID FINITE ELEMENTS USED IN THE RESERVOIR FINITE ELEMENT MODEL

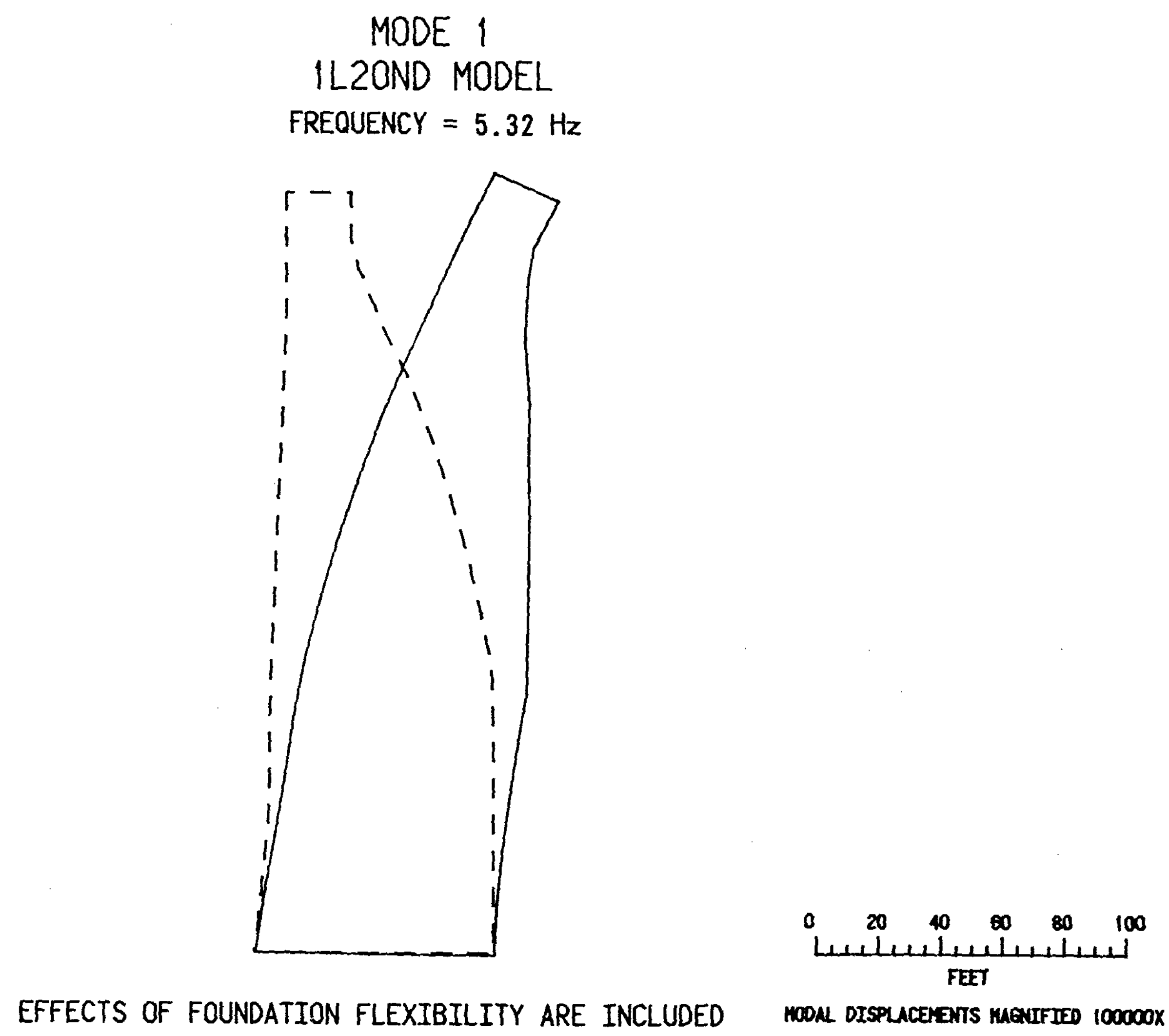


FIGURE 7-1 PLOT OF MODE SHAPE OF CROWN CANTILEVER

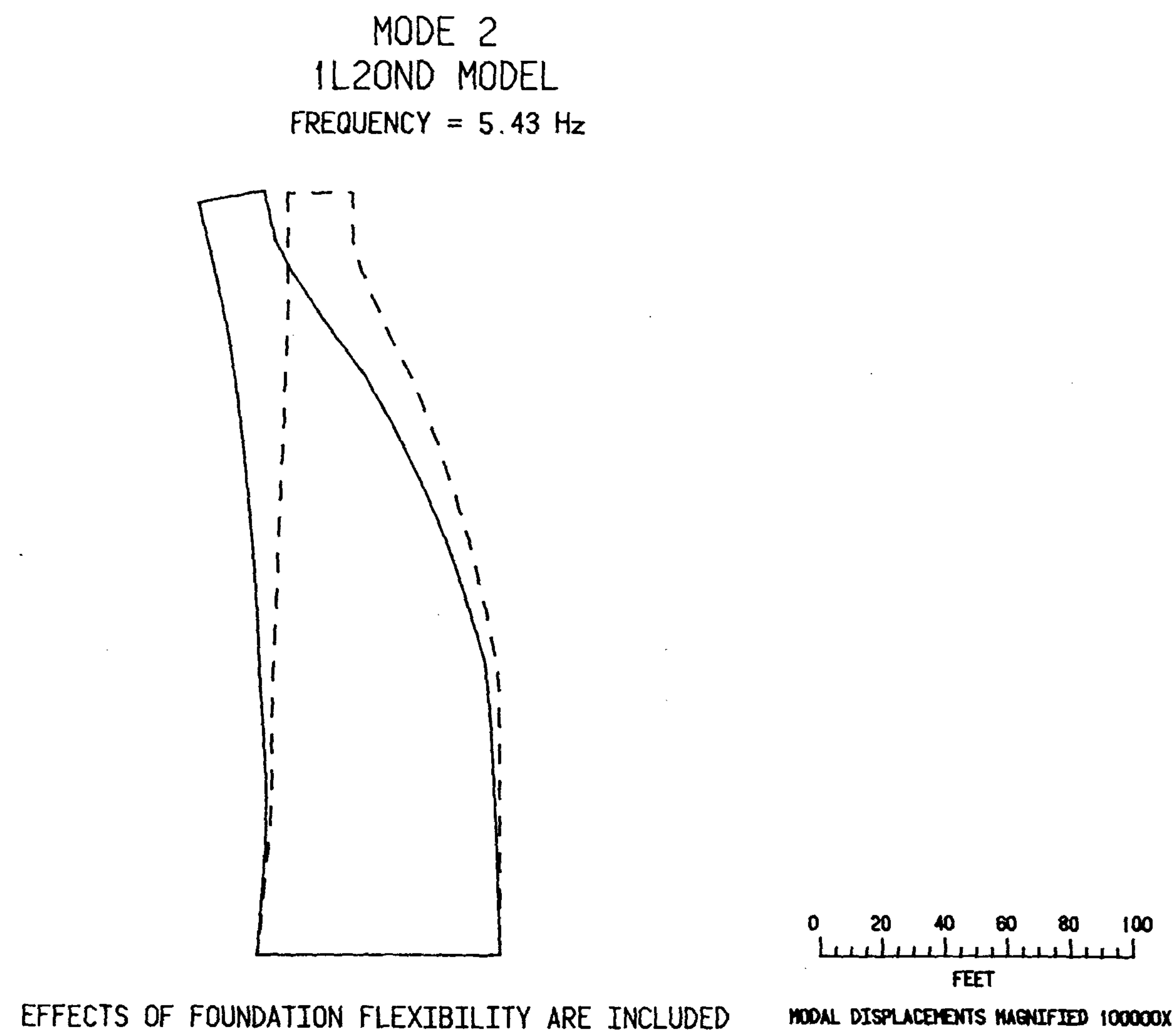


FIGURE 7-2 PLOT OF MODE SHAPE OF CROWN CANTILEVER

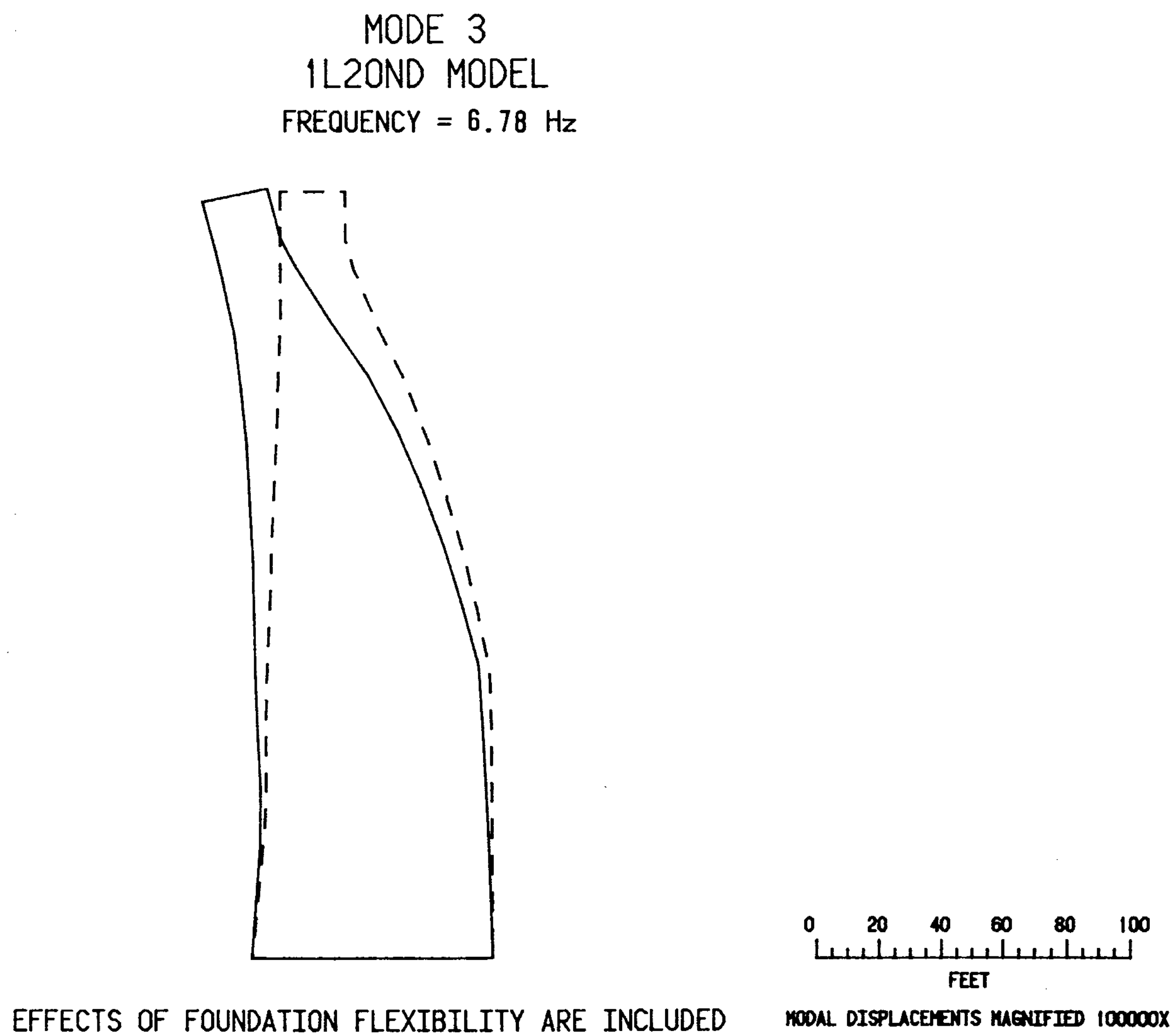


FIGURE 7-3 PLOT OF MODE SHAPE OF CROWN CANTILEVER

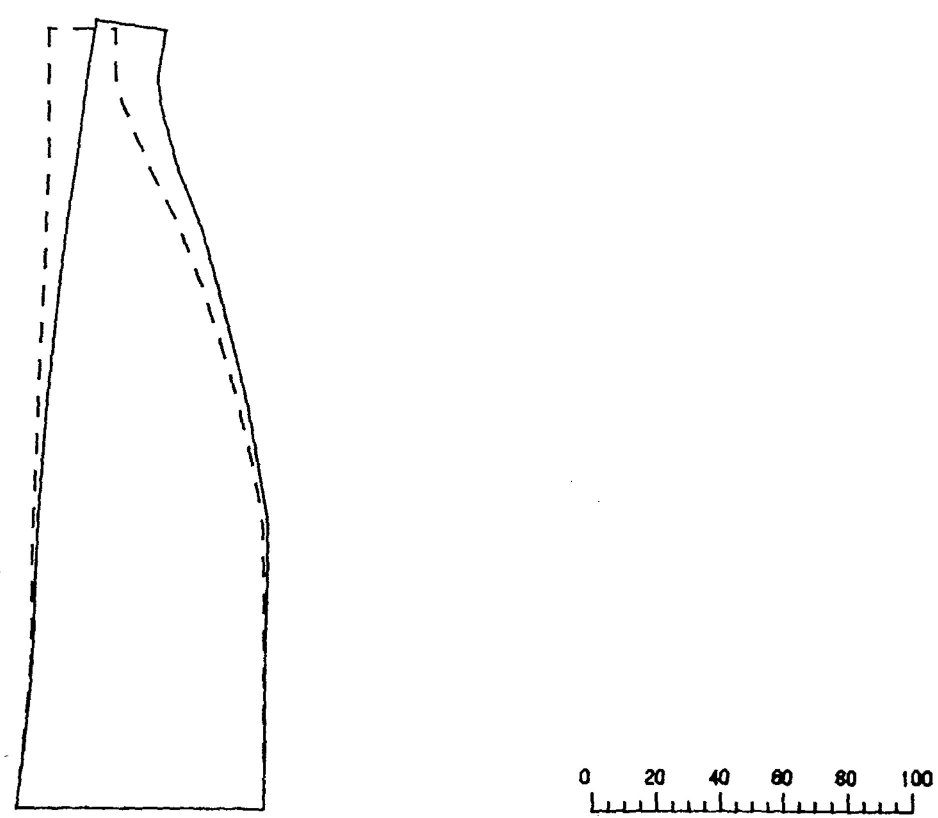


FIGURE 7-4 PLOT OF MODE SHAPE OF CROWN CANTILEVER

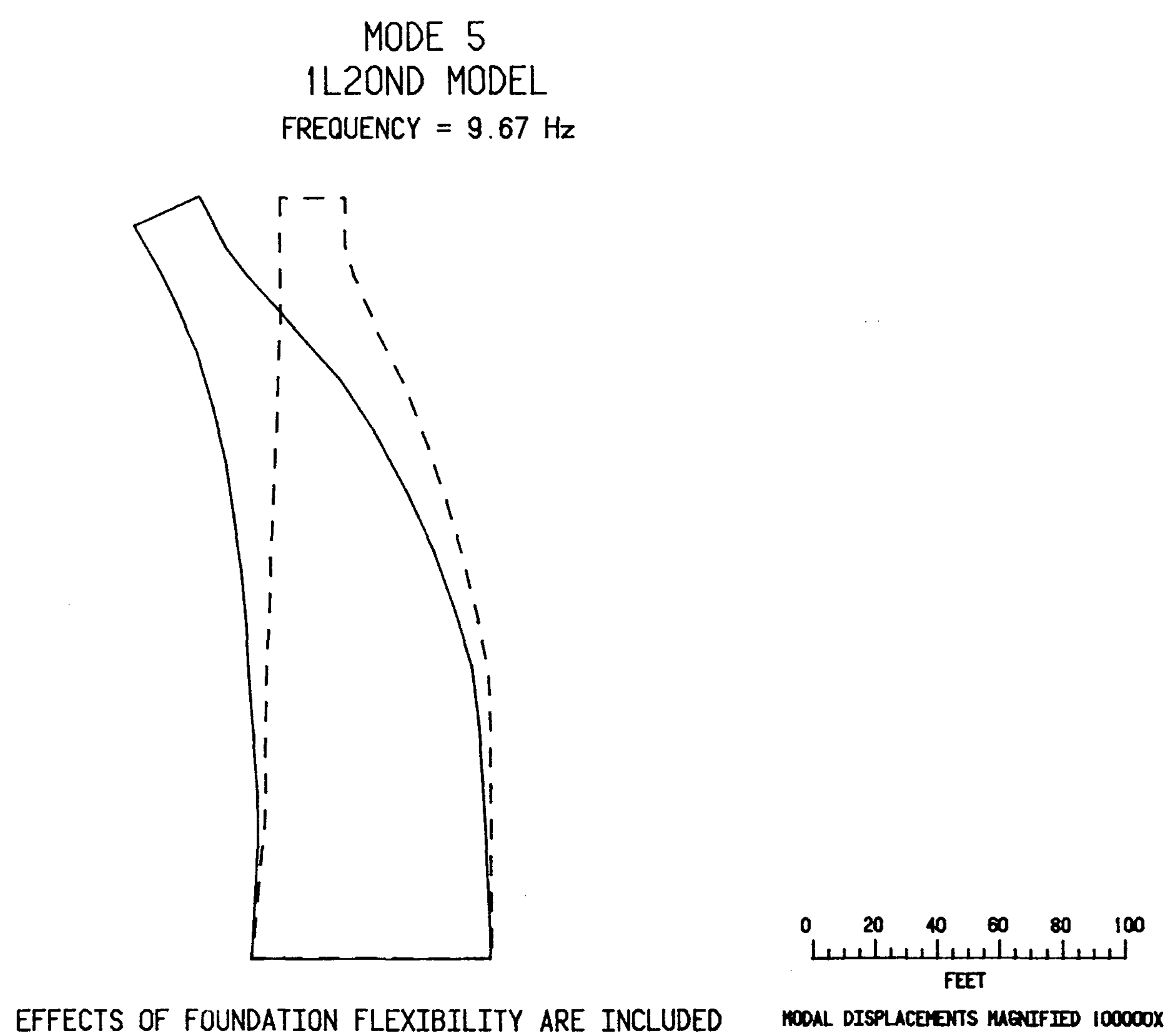
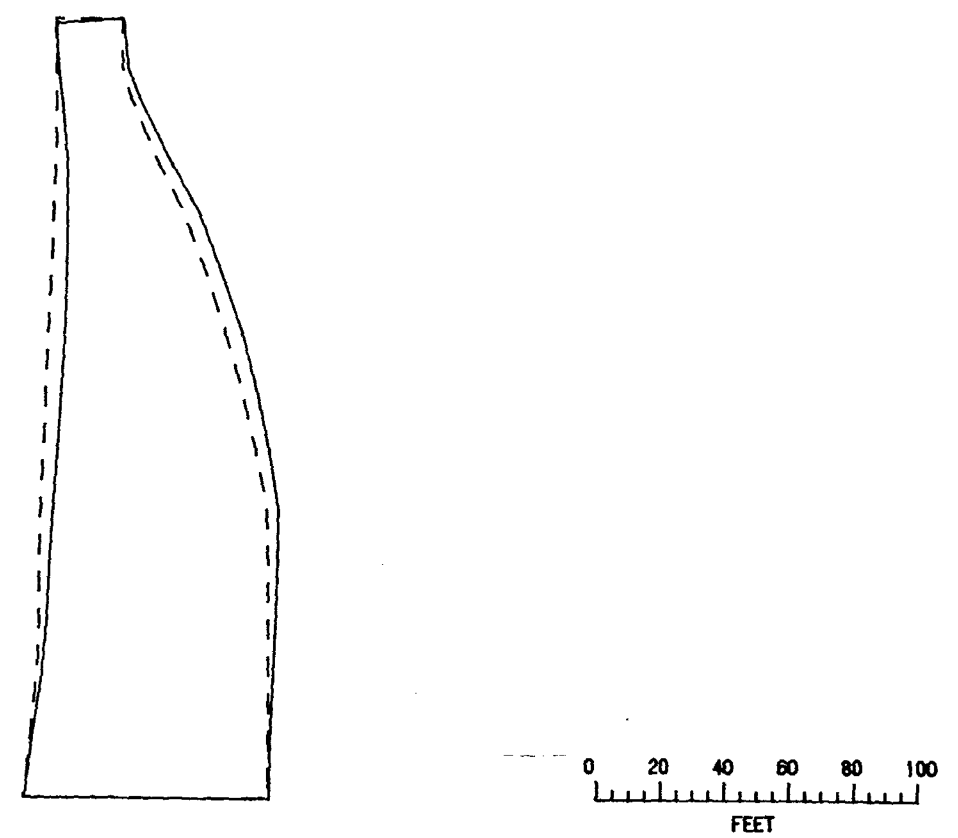


FIGURE 7-5 PLOT OF MODE SHAPE OF CROWN CANTILEVER

MODE 6
1L2OND MODEL
FREQUENCY = 11.59 Hz



EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-6 PLOT OF MODE SHAPE OF CROWN CANTILEVER

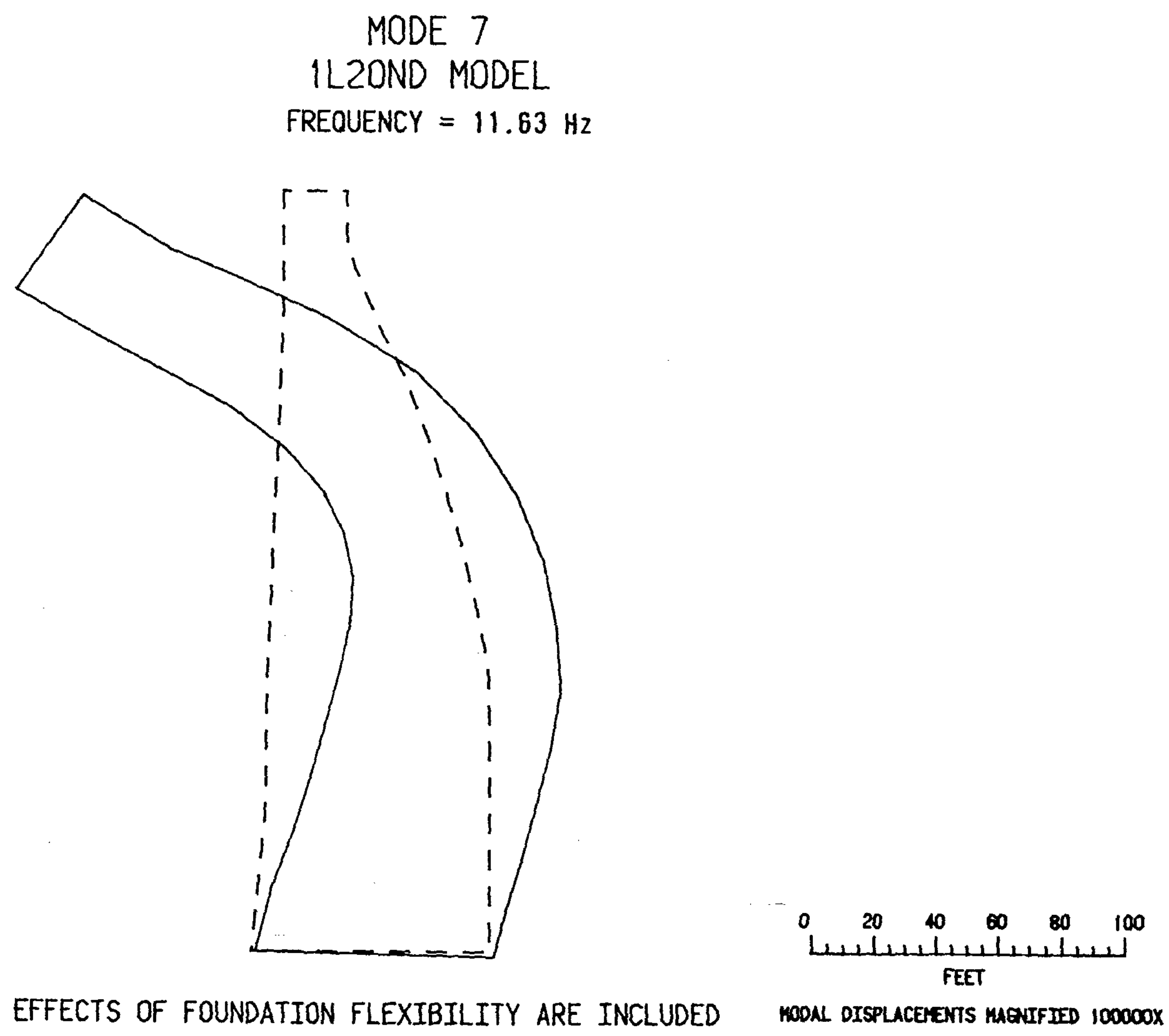
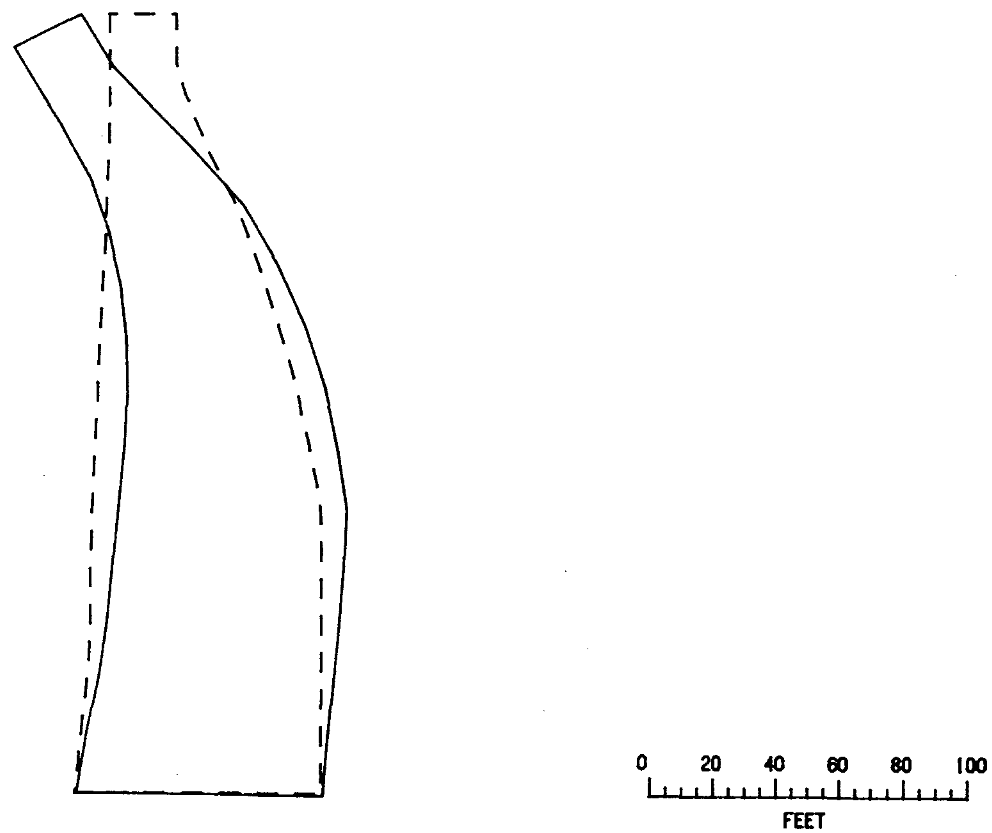


FIGURE 7-7 PLOT OF MODE SHAPE OF CROWN CANTILEVER

MODE 8
1L2OND MODEL
FREQUENCY = 11.94 Hz



EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

MODAL DISPLACEMENTS MAGNIFIED 100000X

FIGURE 7-8 PLOT OF MODE SHAPE OF CROWN CANTILEVER

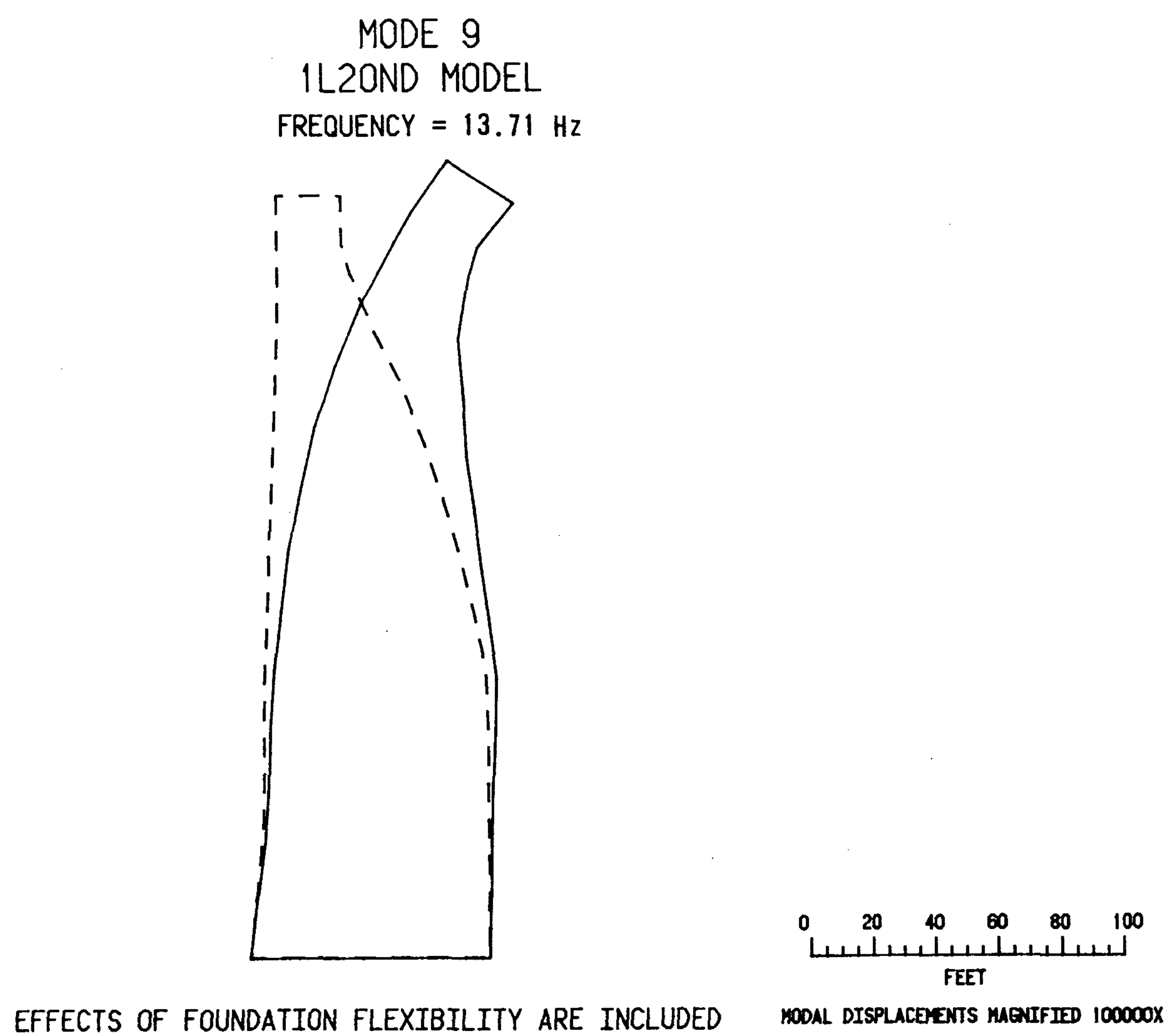
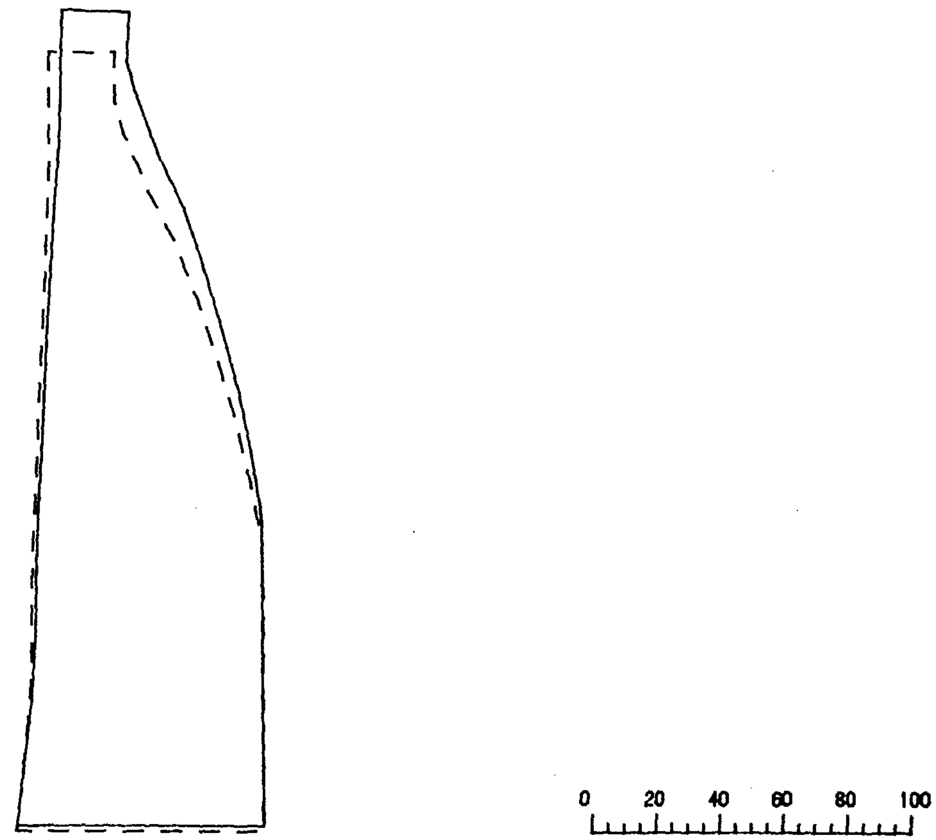


FIGURE 7-9 PLOT OF MODE SHAPE OF CROWN CANTILEVER

MODE 10
1L2OND MODEL
FREQUENCY = 14.29 Hz



EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-10 PLOT OF MODE SHAPE OF CROWN CANTILEVER

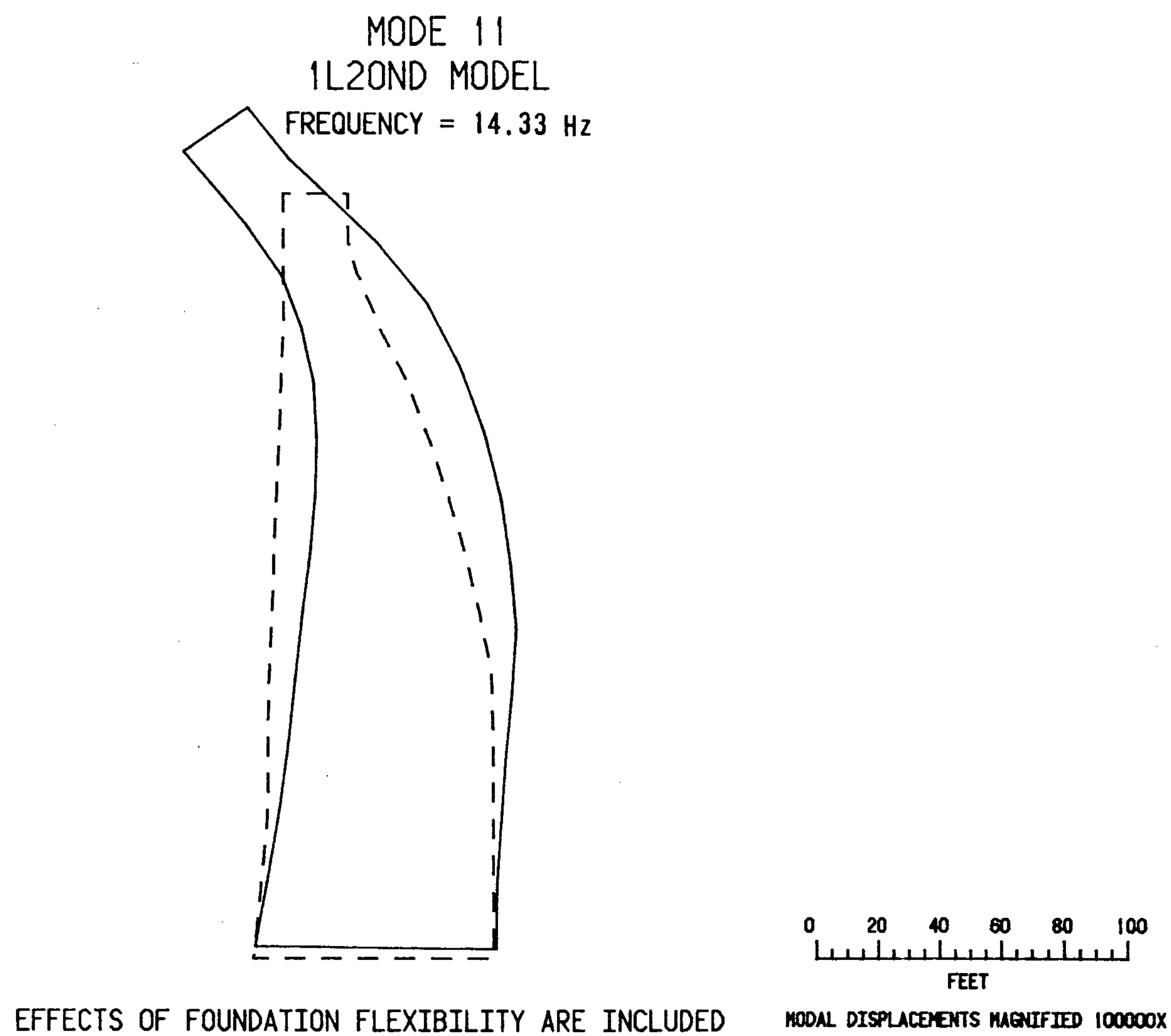


FIGURE 7-11 PLOT OF MODE SHAPE OF CROWN CANTILEVER

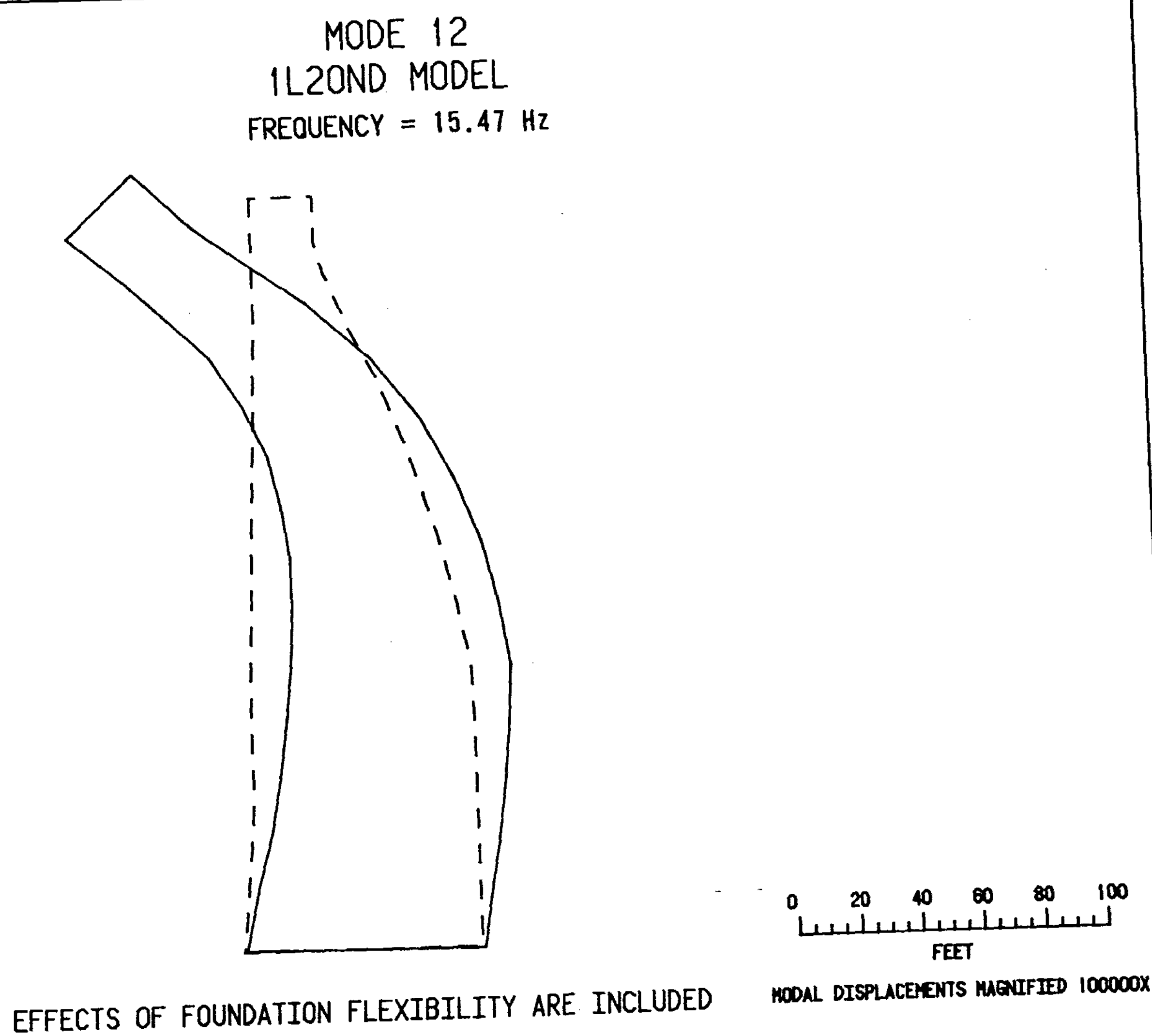
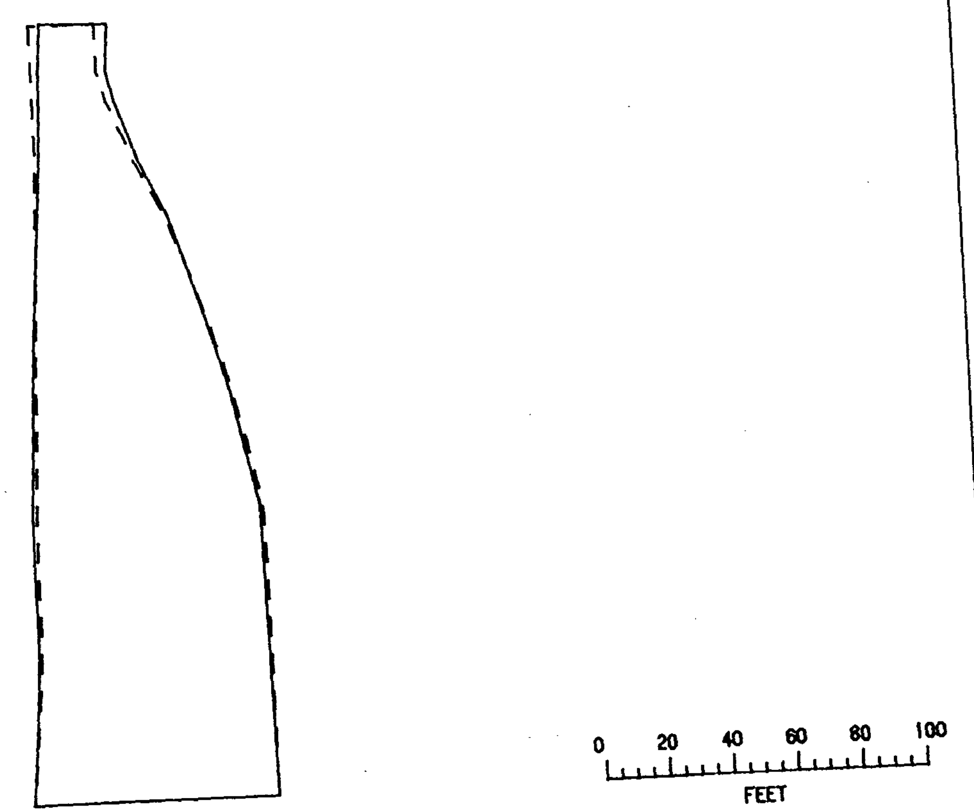


FIGURE 7-12 PLOT OF MODE SHAPE OF CROWN CANTILEVER

MODE 13
1L2OND MODEL
FREQUENCY = 16.08 Hz



EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-13 PLOT OF MODE SHAPE OF CROWN CANTILEVER

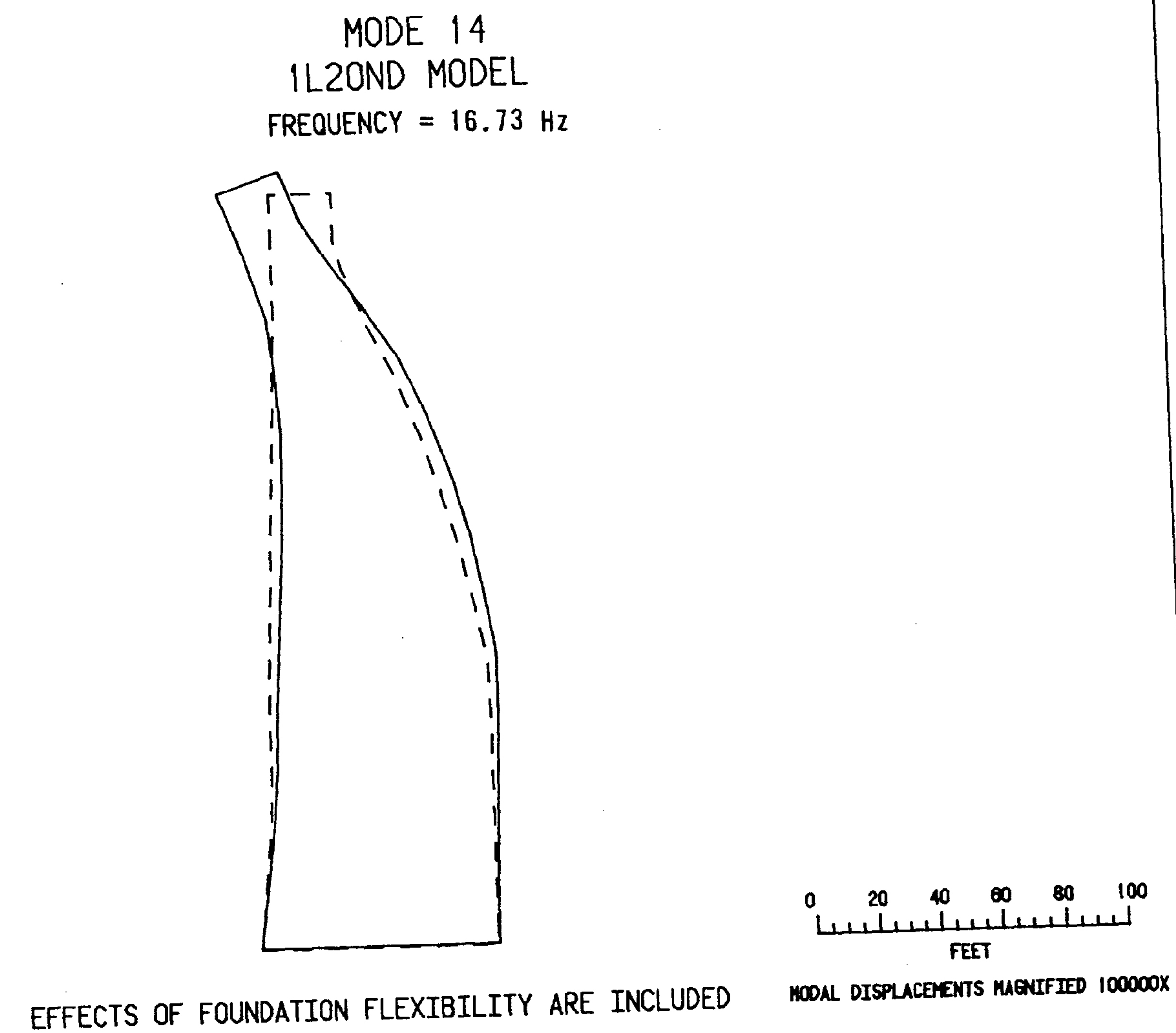


FIGURE 7-14 PLOT OF MODE SHAPE OF CROWN CANTILEVER

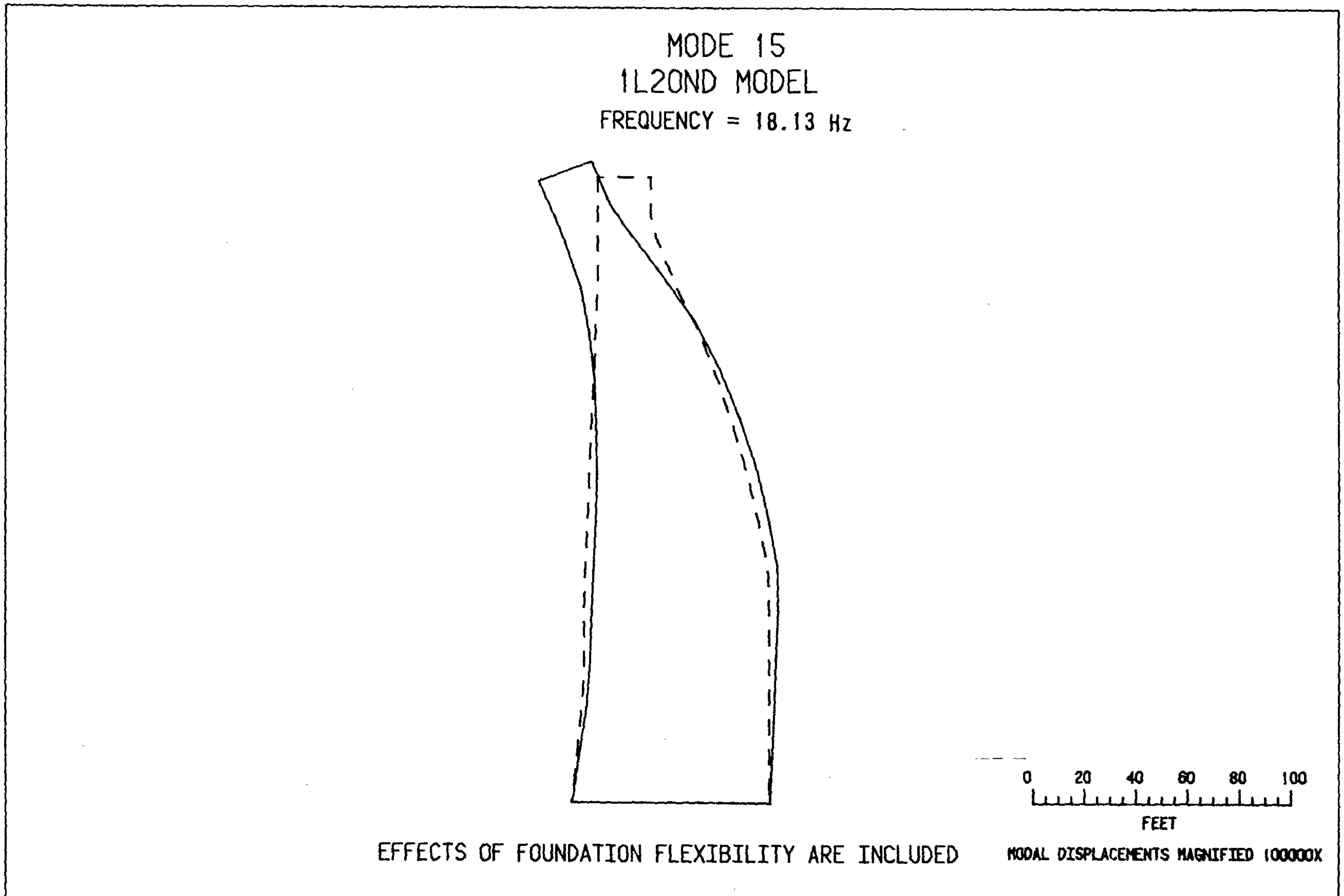


FIGURE 7-15 PLOT OF MODE SHAPE OF CROWN CANTILEVER

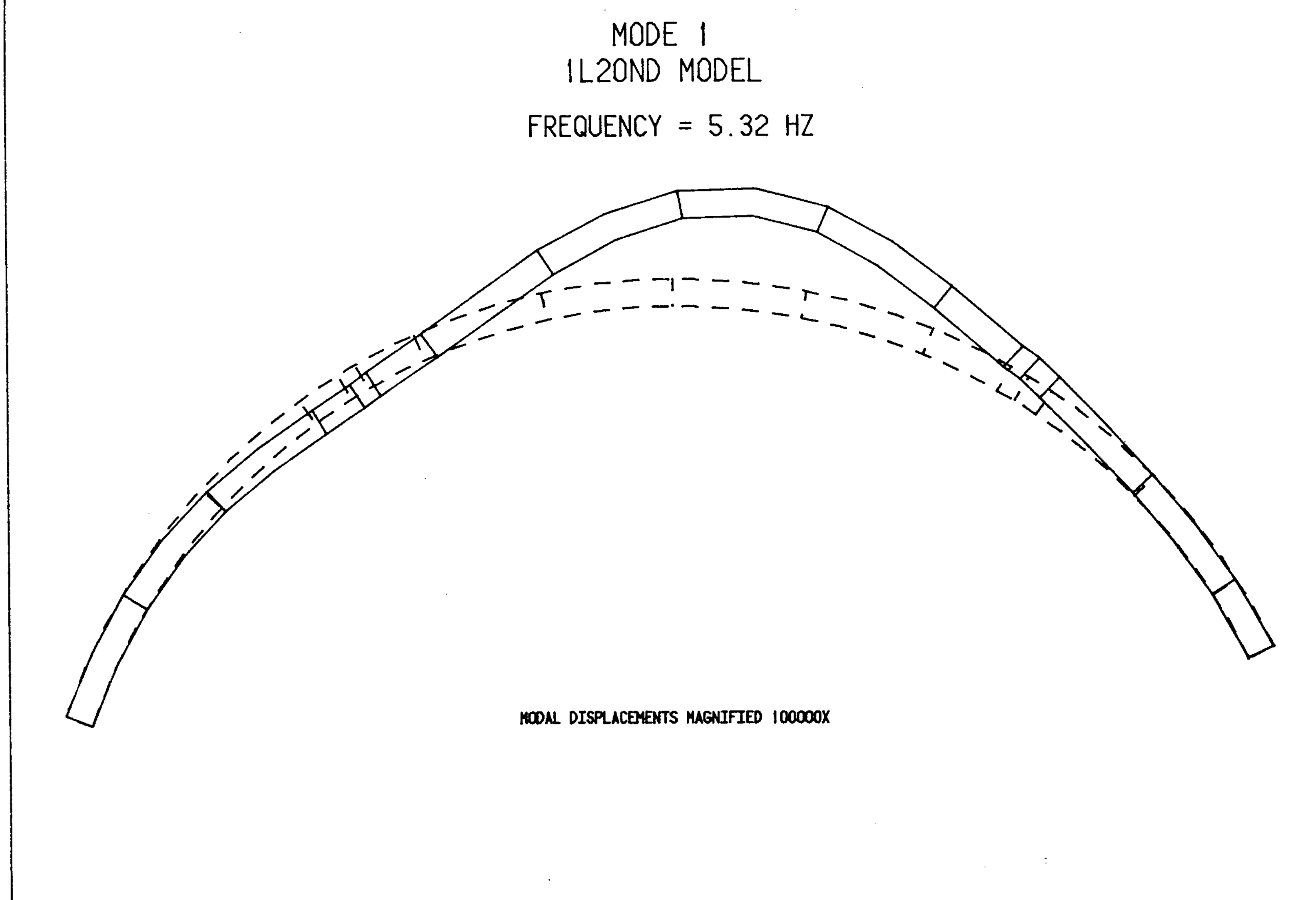


FIGURE 7-16 PLOT OF MODE SHAPE OF CREST

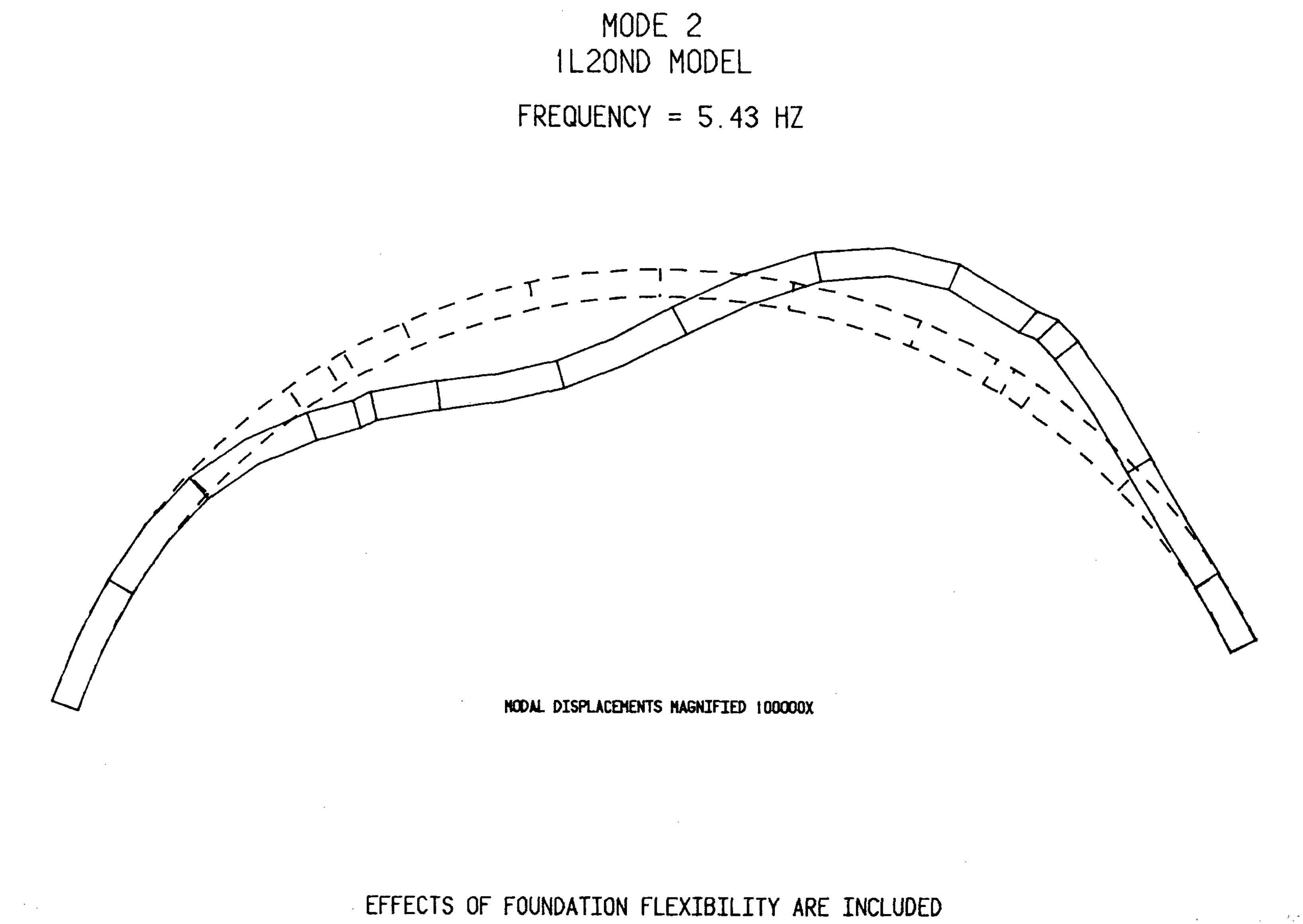
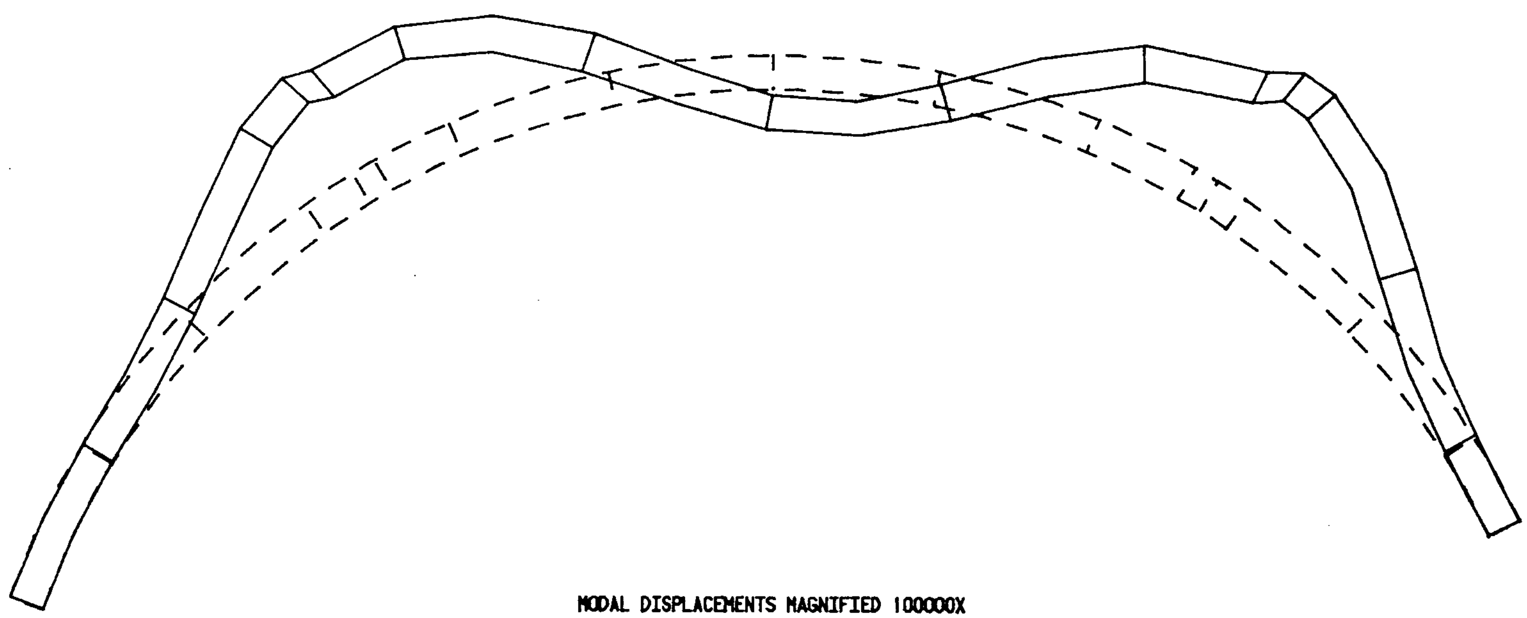


FIGURE 7-17 PLOT OF MODE SHAPE OF CREST

MODE 3
1L2OND MODEL
FREQUENCY = 6.78 HZ

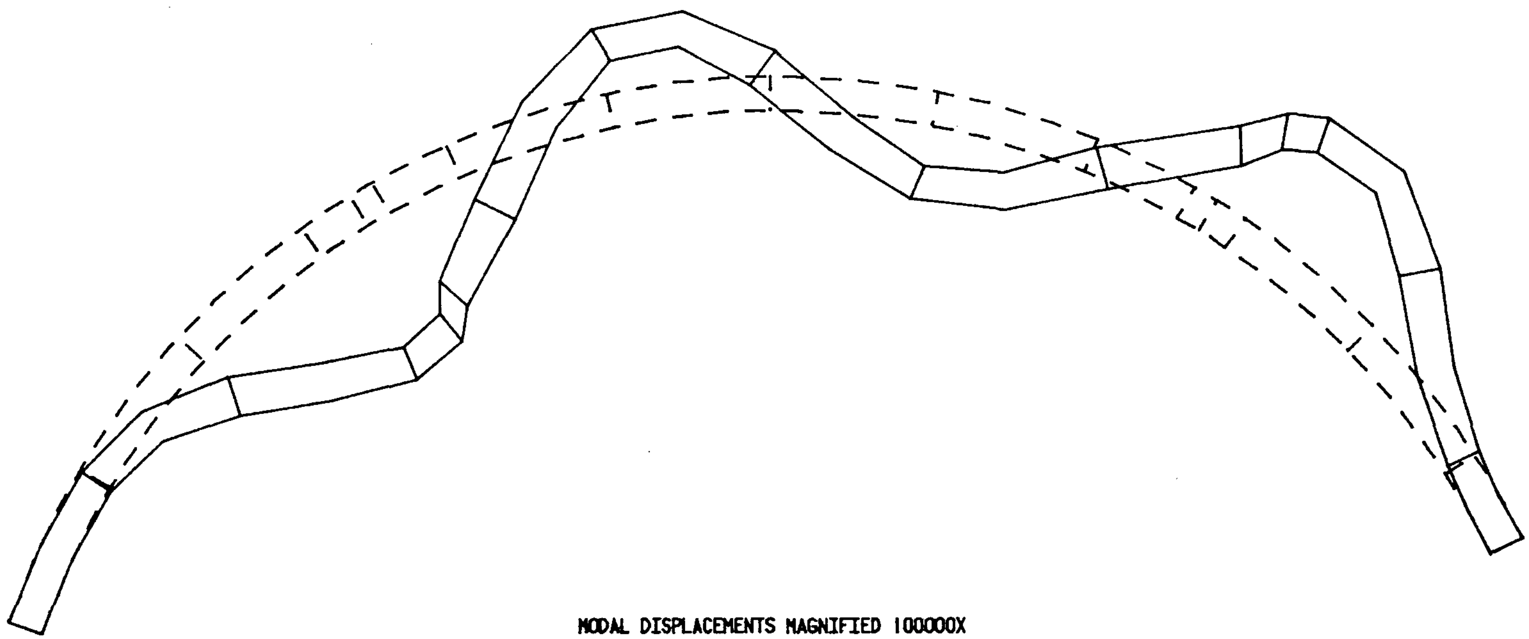


EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-18 PLOT OF MODE SHAPE OF CREST

MODE 4
1L20ND MODEL

FREQUENCY = 8.06 HZ



MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-19 PLOT OF MODE SHAPE OF CREST

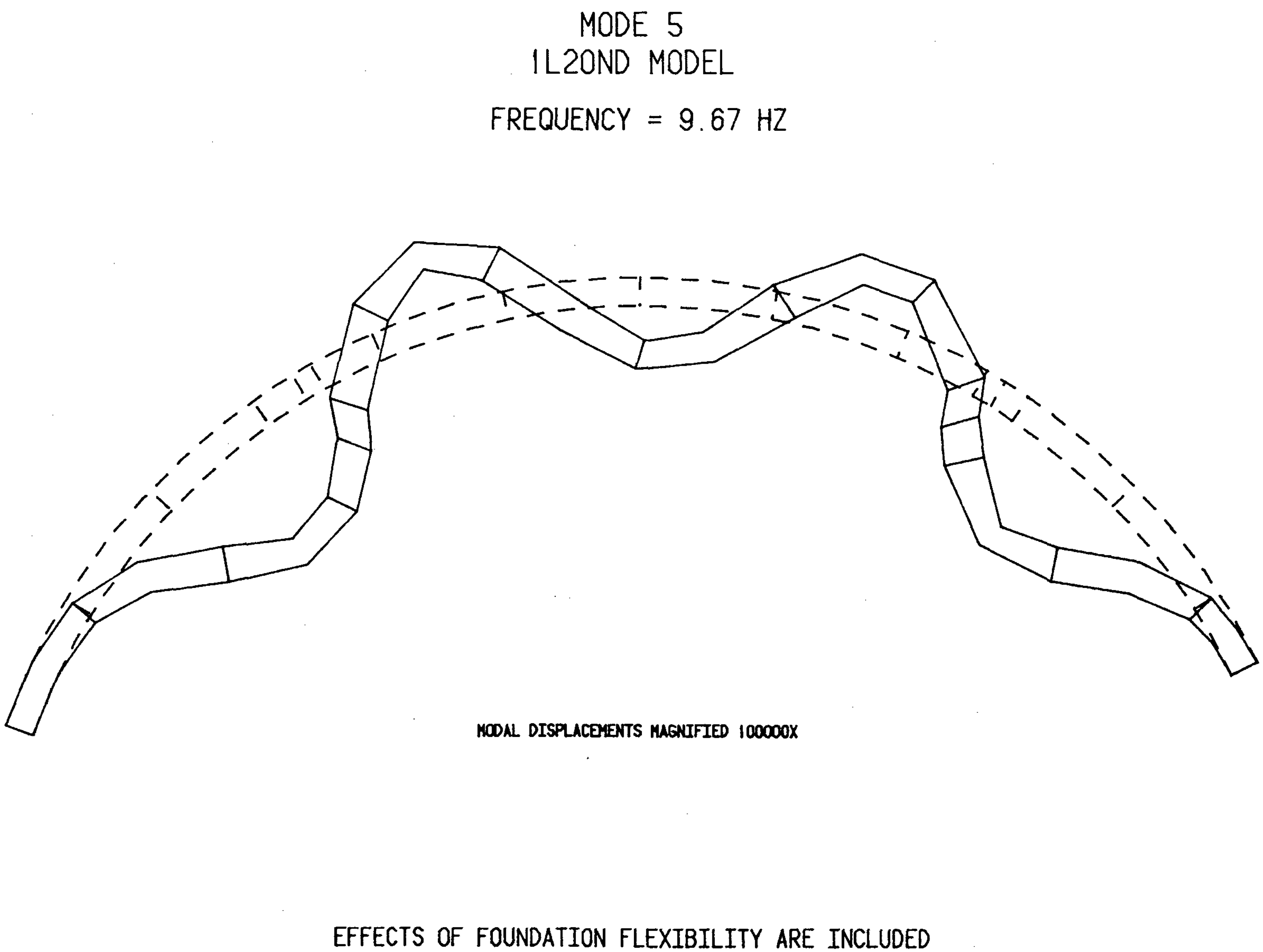
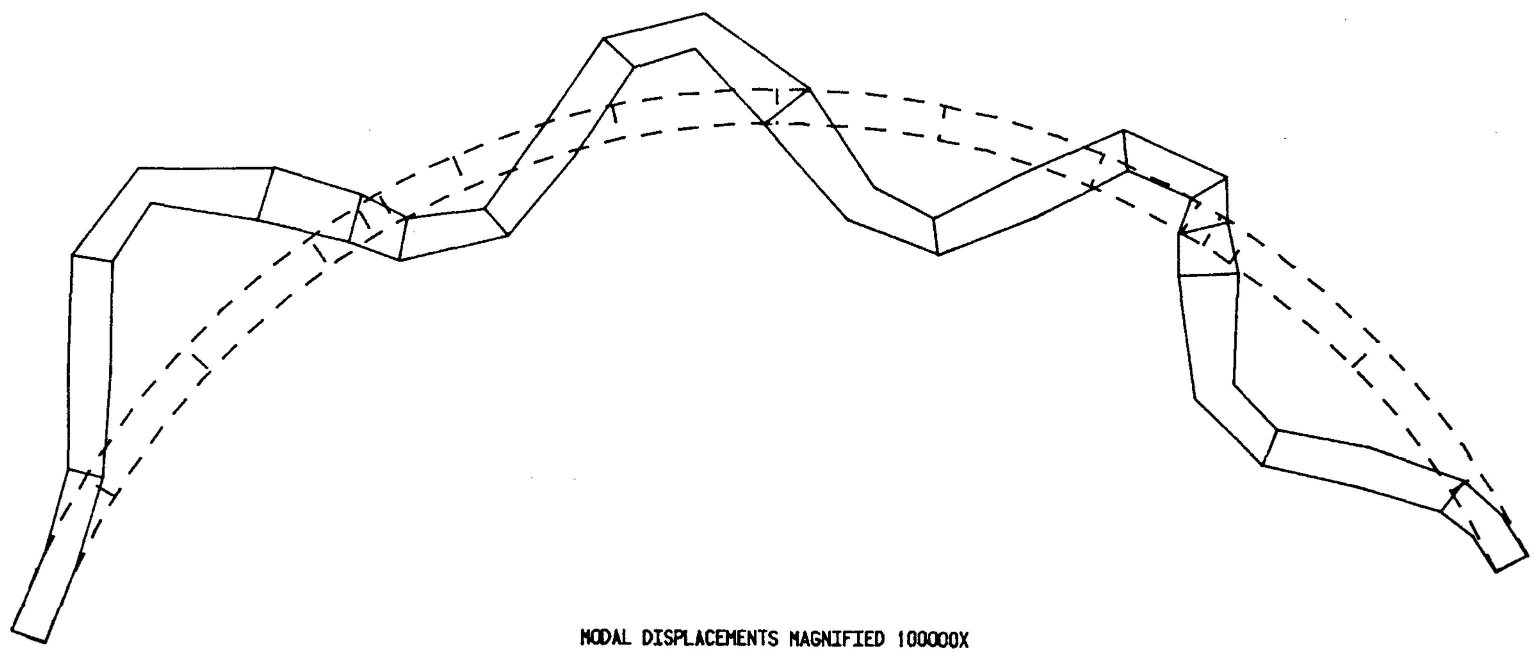


FIGURE 7-20 PLOT OF MODE SHAPE OF CREST

MODE 6
1L2OND MODEL

FREQUENCY = 11.59 HZ

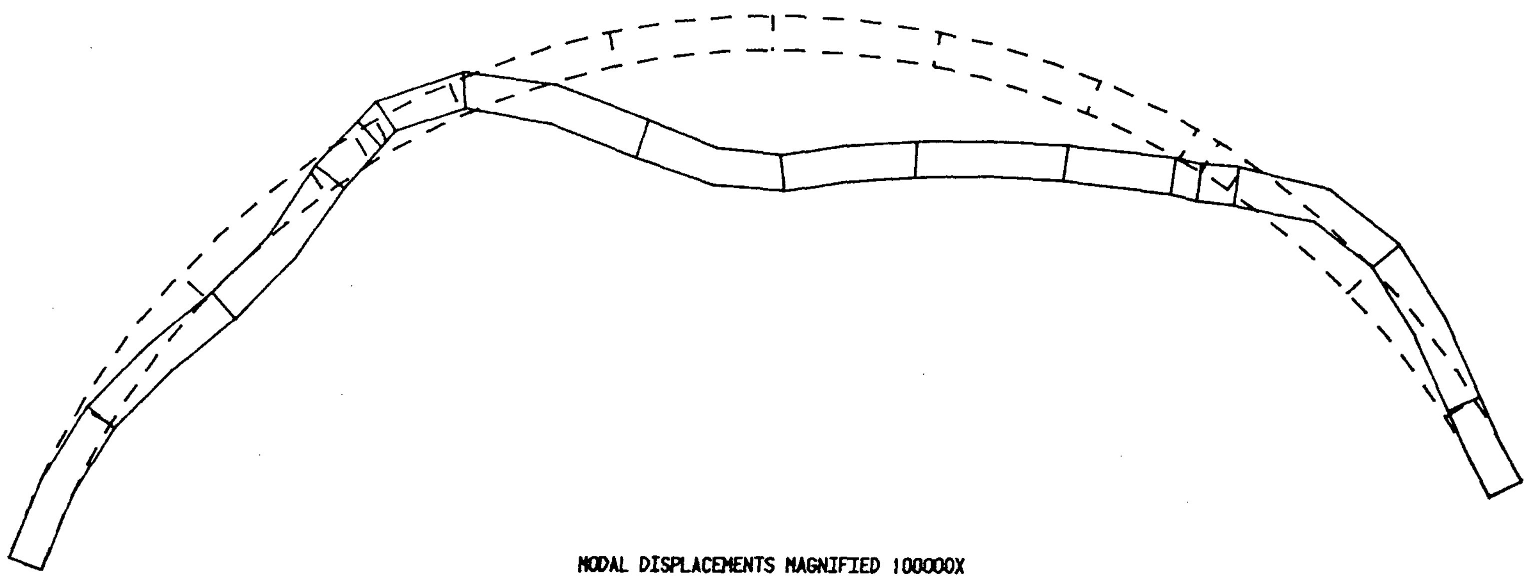


EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-21 PLOT OF MODE SHAPE OF CREST

MODE 7
1L2OND MODEL

FREQUENCY = 11.63 HZ

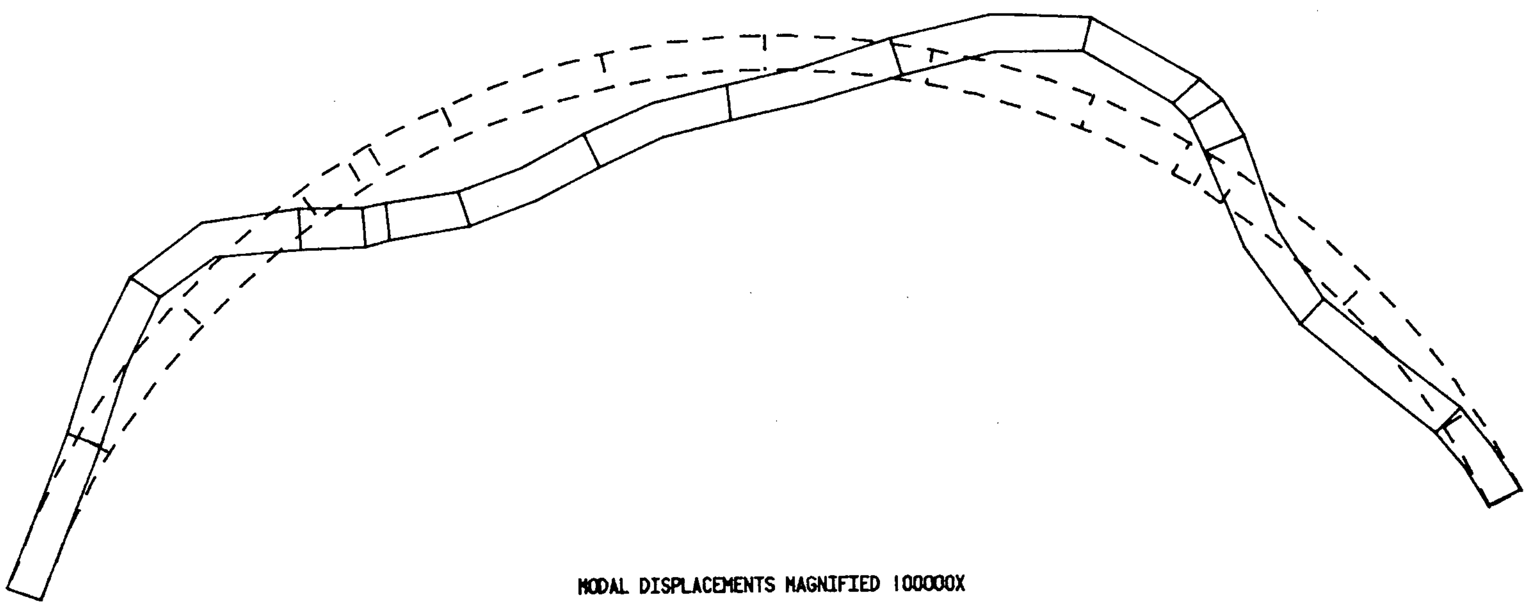


EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-22 PLOT OF MODE SHAPE OF CREST

MODE 8
1L2OND MODEL

FREQUENCY = 11.94 HZ



MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-23 PLOT OF MODE SHAPE OF CREST

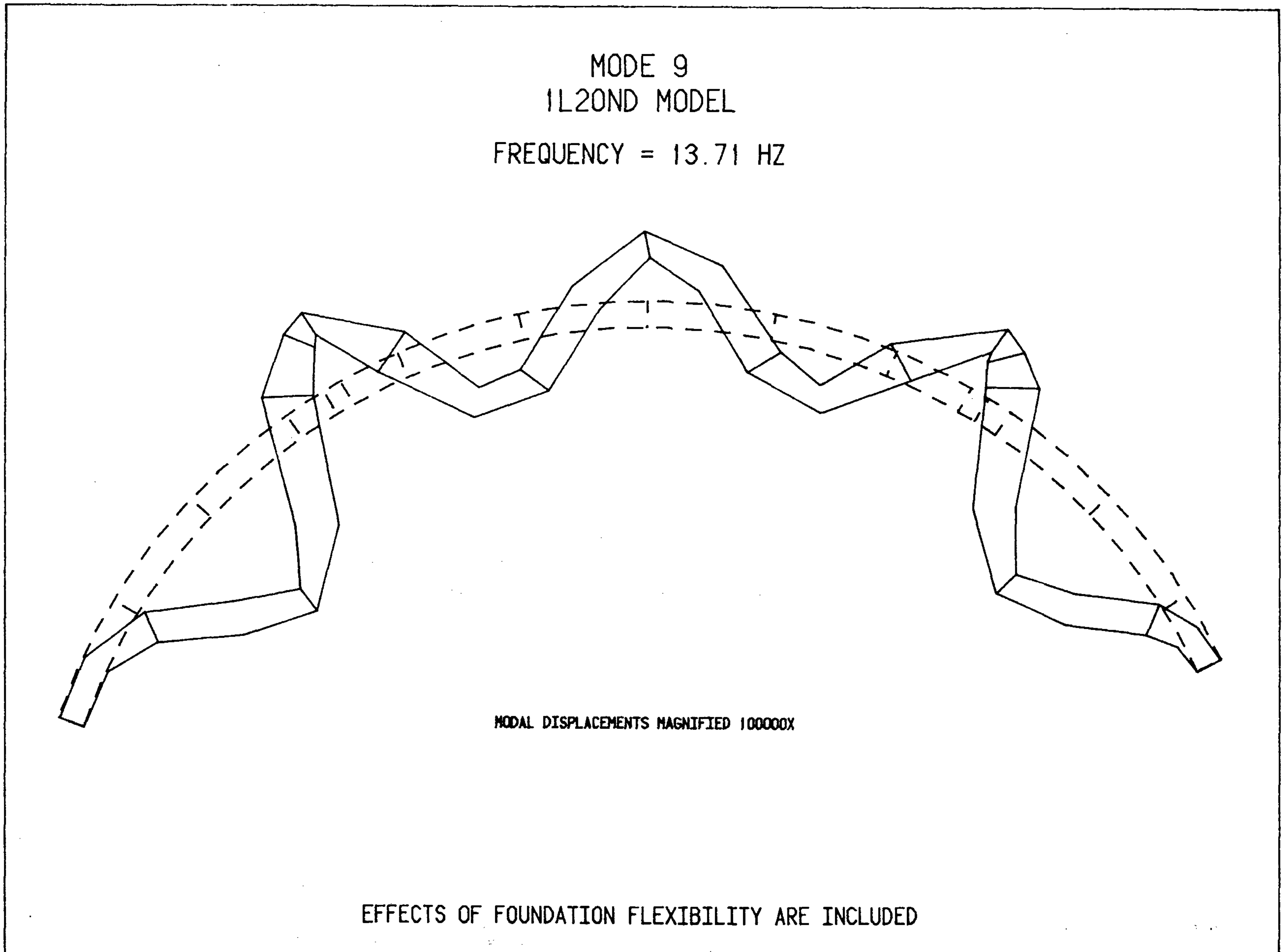
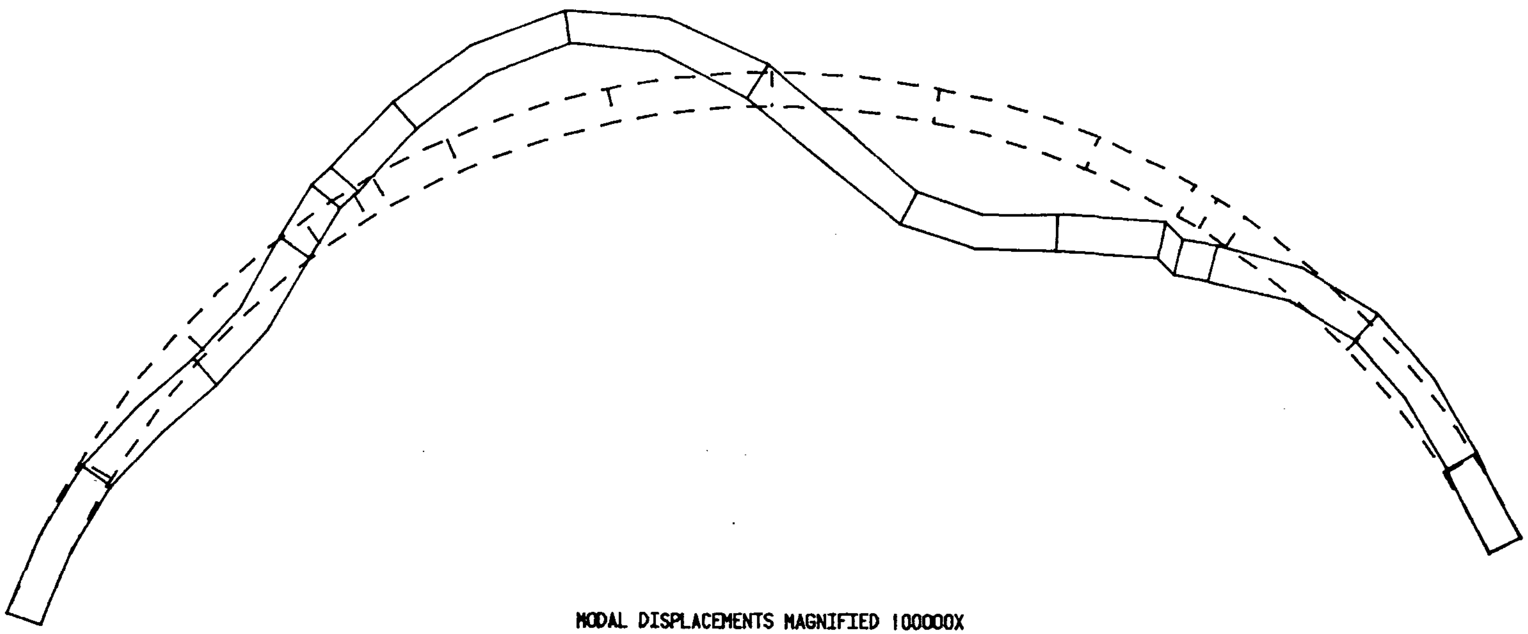


FIGURE 7-24 PLOT OF MODE SHAPE OF CREST

MODE 10
1L20ND MODEL

FREQUENCY = 14.29 HZ



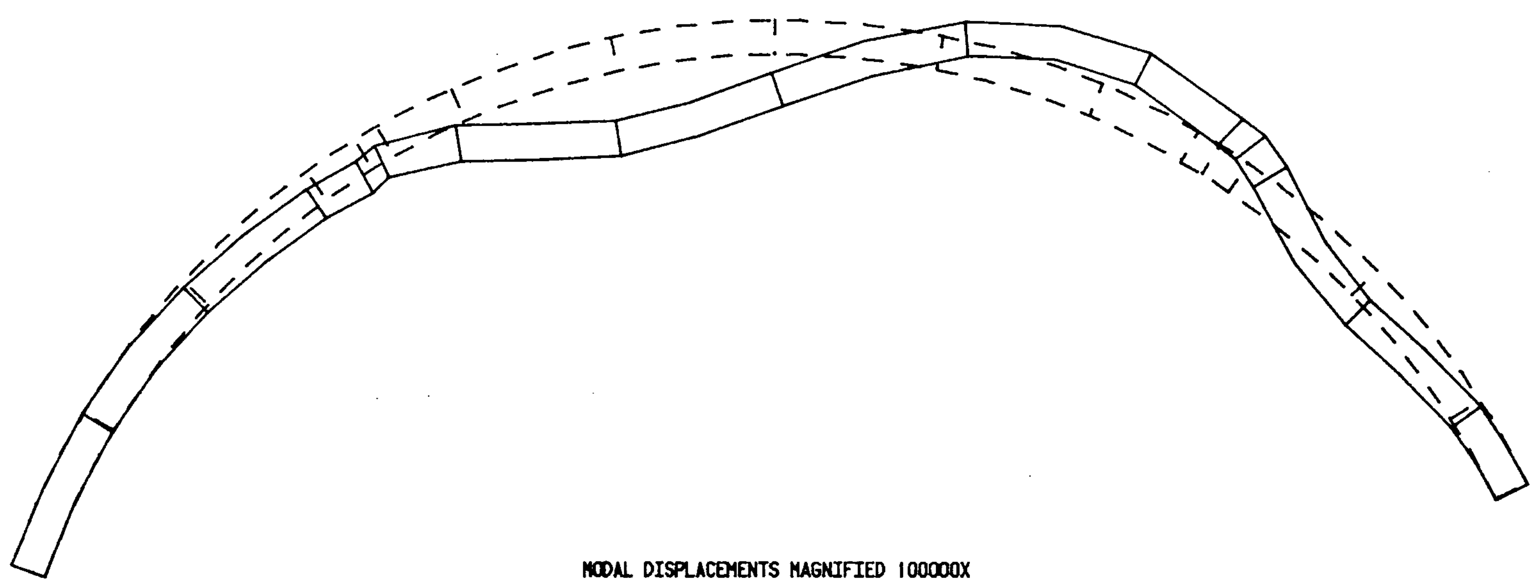
MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-25 PLOT OF MODE SHAPE OF CREST

MODE 11
1L20ND MODEL

FREQUENCY = 14.33 HZ



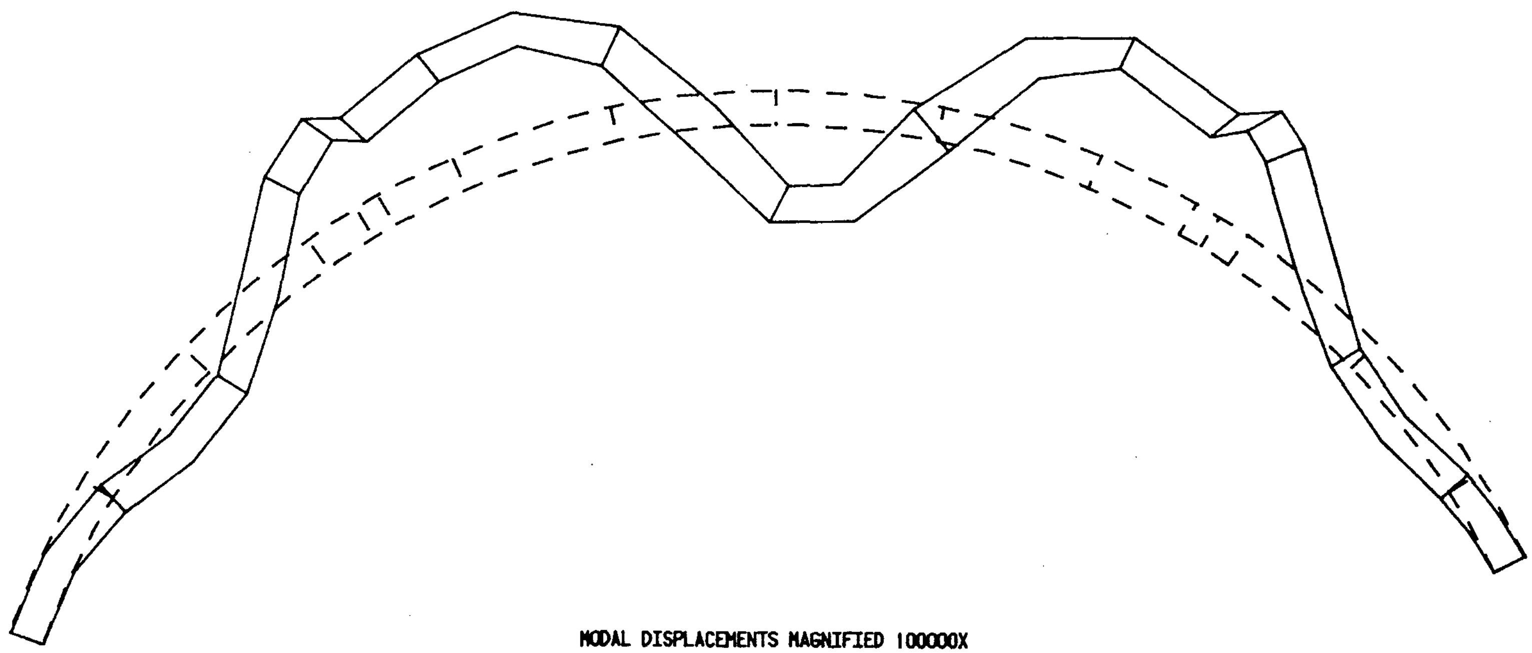
MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-26 PLOT OF MODE SHAPE OF CREST

MODE 12
1L2OND MODEL

FREQUENCY = 15.47 HZ

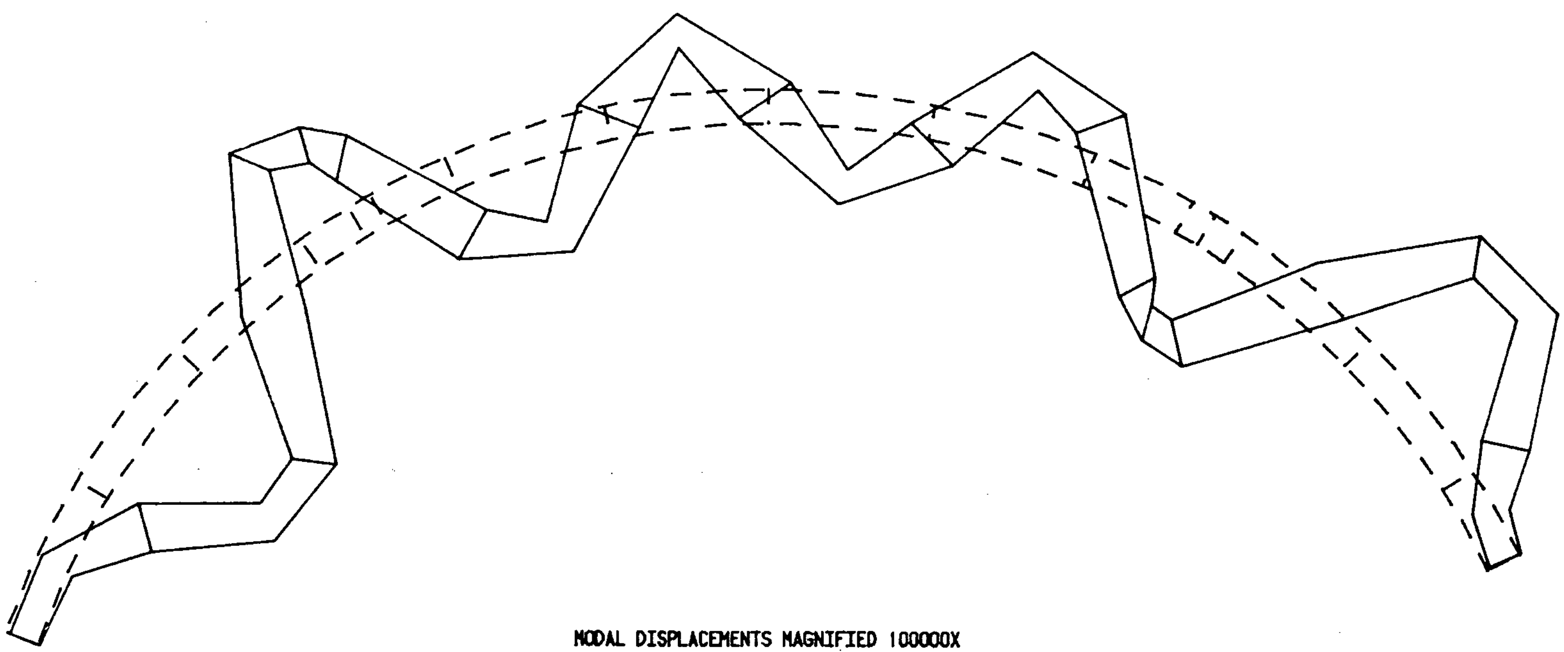


EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-27 PLOT OF MODE SHAPE OF CREST

MODE 13
1L2OND MODEL

FREQUENCY = 16.08 HZ



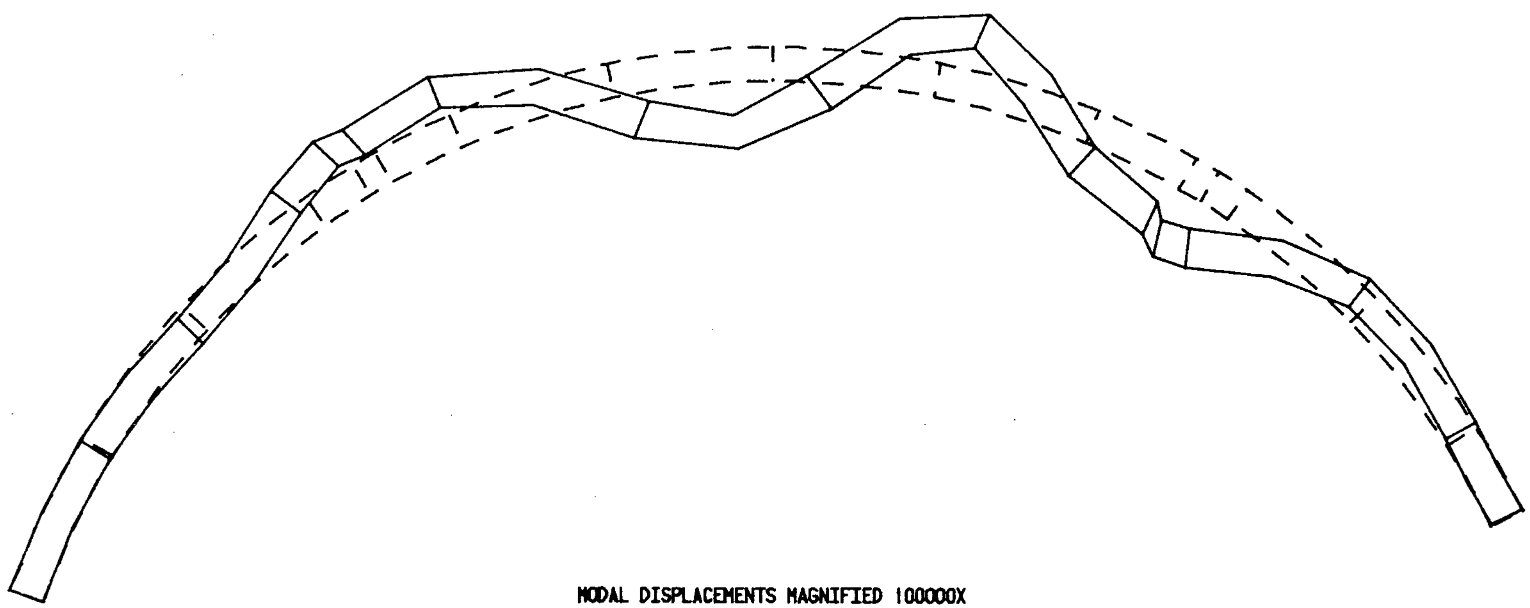
MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-28 PLOT OF MODE SHAPE OF CREST

MODE 14
1L20ND MODEL

FREQUENCY = 16.73 HZ

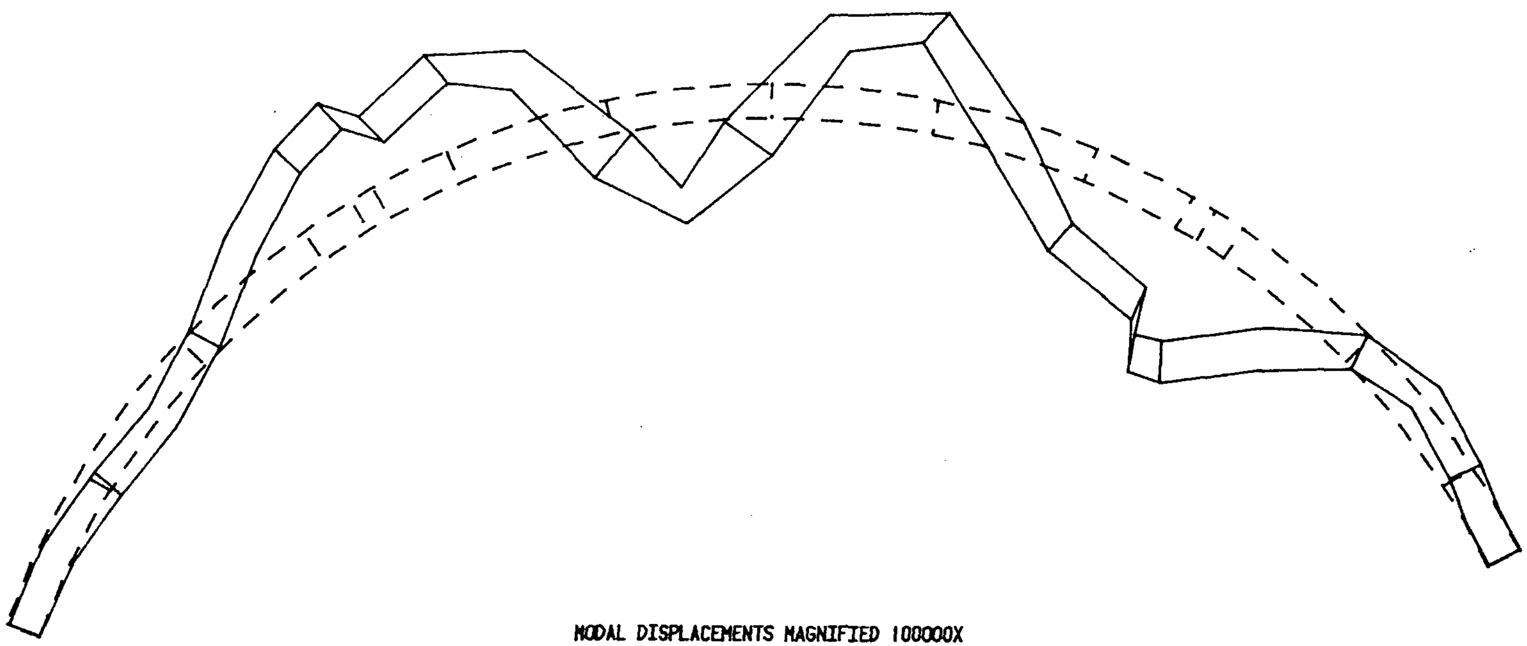


EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-29 PLOT OF MODE SHAPE OF CREST

MODE 15
1L20ND MODEL

FREQUENCY = 18.13 HZ



MODAL DISPLACEMENTS MAGNIFIED 100000X

EFFECTS OF FOUNDATION FLEXIBILITY ARE INCLUDED

FIGURE 7-30 PLOT OF MODE SHAPE OF CREST

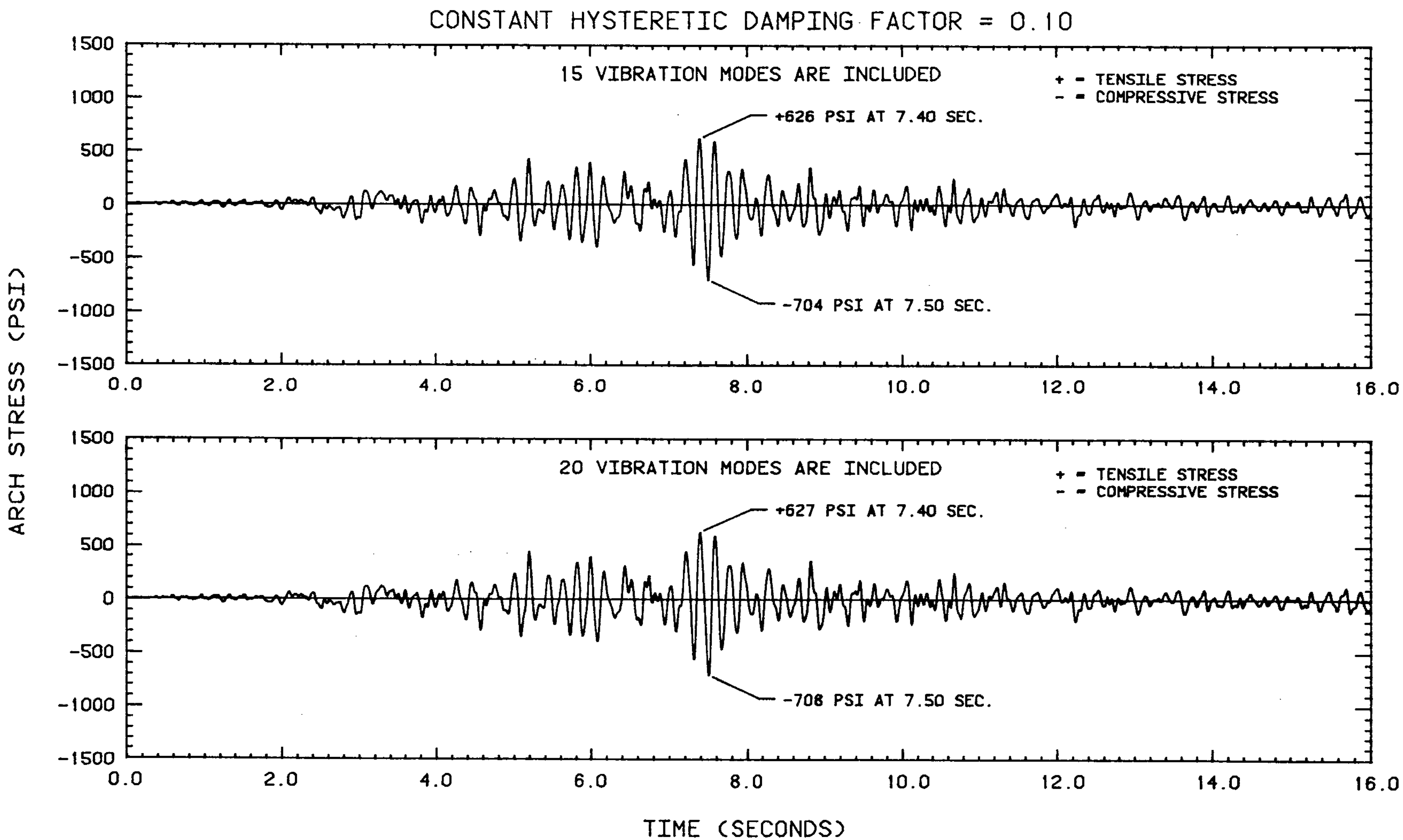


FIGURE 7-31 DYNAMIC ARCH STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 1. RESPONSE IS COMPUTED FOR THE DAM ON RIGID FOUNDATION ROCK WITH EMPTY RESERVOIR. STATIC STRESSES ARE NOT INCLUDED. CONSTANT HYSTERETIC DAMPING FACTOR = 0.10

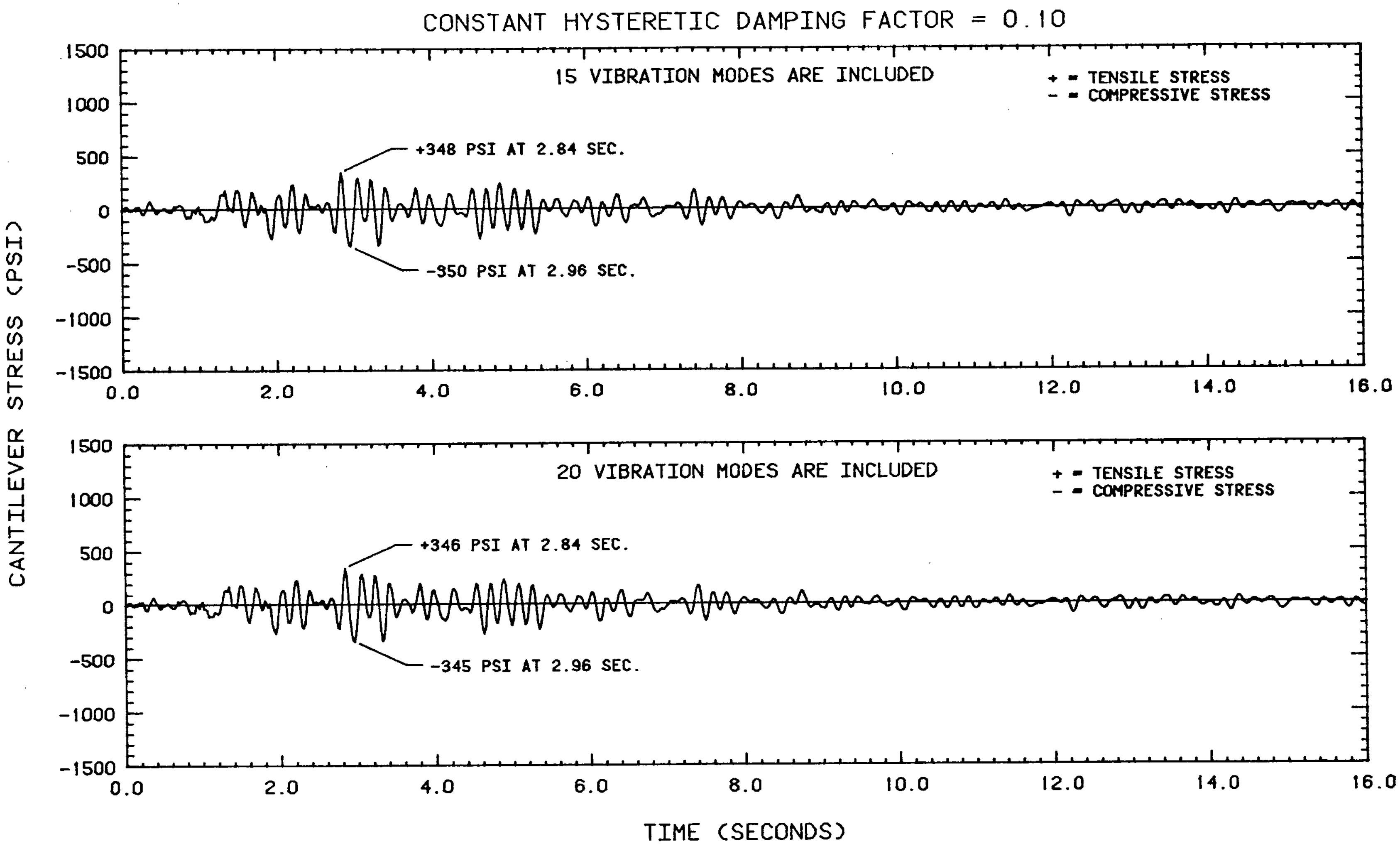


FIGURE 7-32 DYNAMIC CANTILEVER STRESS RESPONSE AT STRESS LOCATION 3 IN ELEMENT 51 DUE TO QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON RIGID FOUNDATION ROCK WITH EMPTY RESERVOIR. STATIC STRESSES ARE NOT INCLUDED. CONSTANT HYSTERETIC DAMPING FACTOR = 0.10

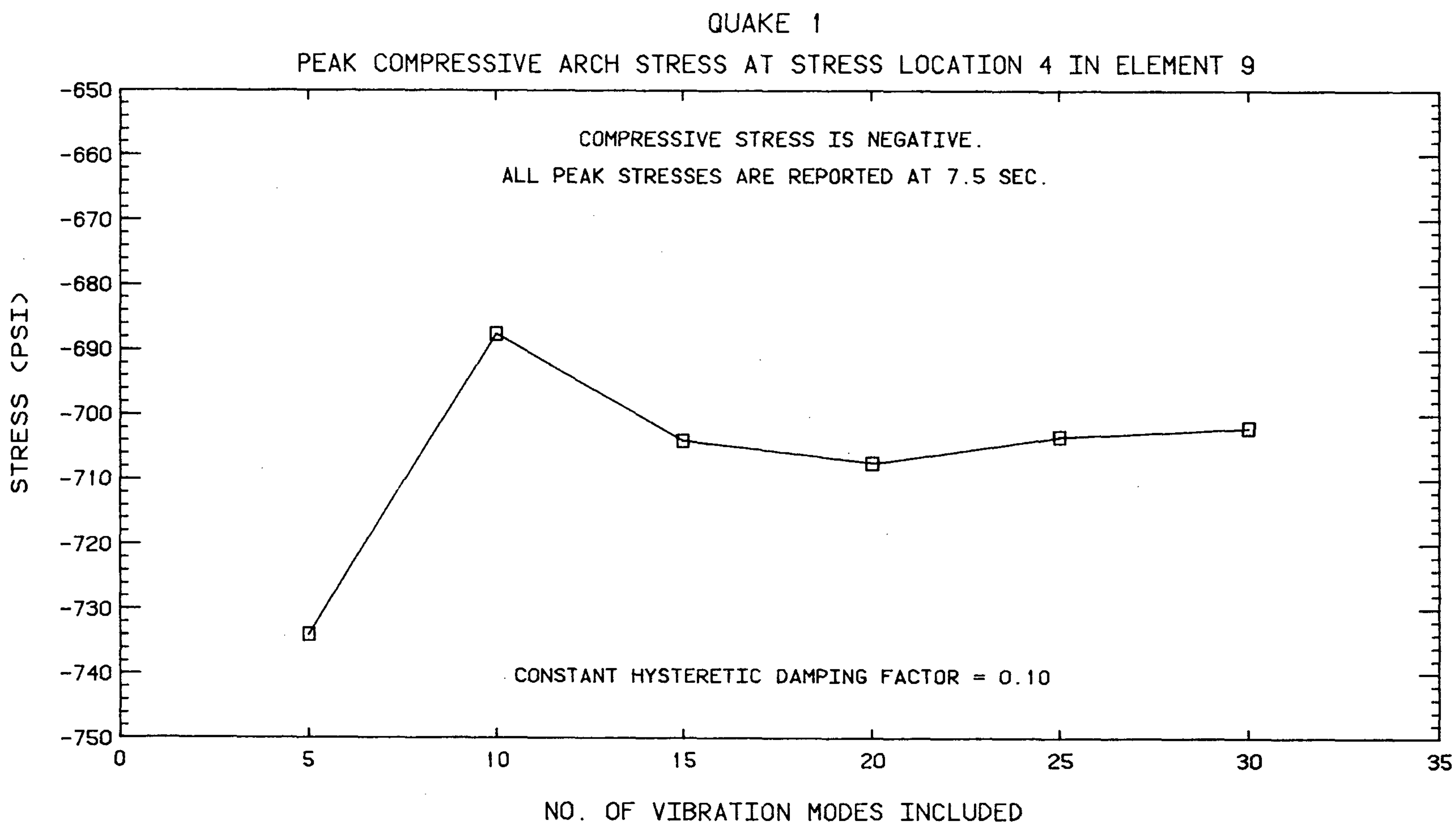


FIGURE 7-33 PLOT OF PEAK (DYNAMIC) COMPRESSIVE ARCH STRESS AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 1 FOR SELECTED NUMBERS OF VIBRATION MODES. STATIC STRESSES ARE NOT INCLUDED. THE DAM IS ASSUMED TO BE SUPPORTED ON RIGID FOUNDATION ROCK WITH AN EMPTY RESERVOIR.

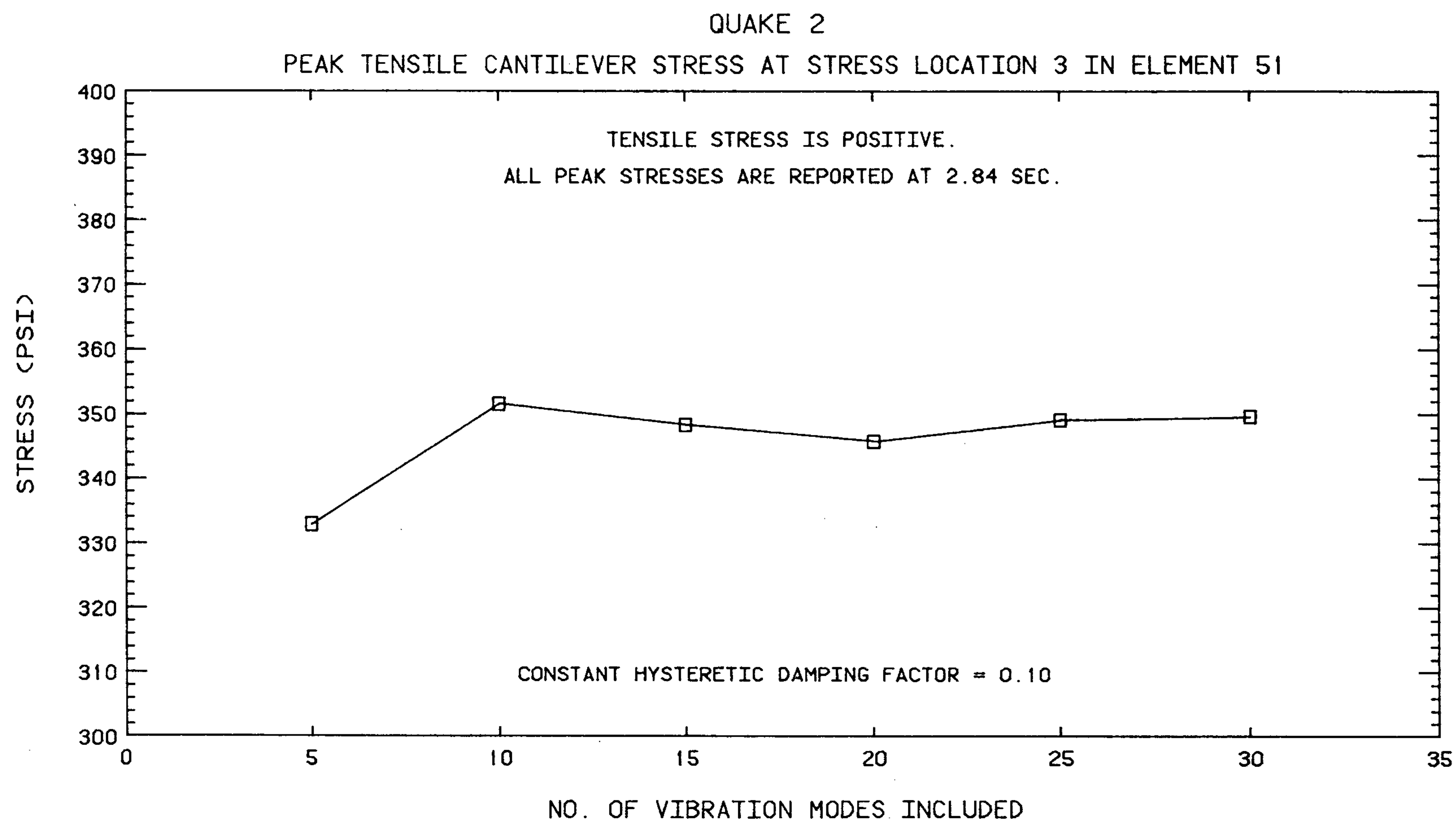


FIGURE 7-34 PLOT OF PEAK (DYNAMIC) TENSILE CANTILEVER STRESS AT STRESS LOCATION 3 IN ELEMENT 51 DUE TO QUAKE 2 FOR SELECTED NUMBERS OF VIBRATION MODES. STATIC STRESSES ARE NOT INCLUDED. THE DAM IS ASSUMED TO BE SUPPORTED ON RIGID FOUNDATION ROCK WITH AN EMPTY RESERVOIR.

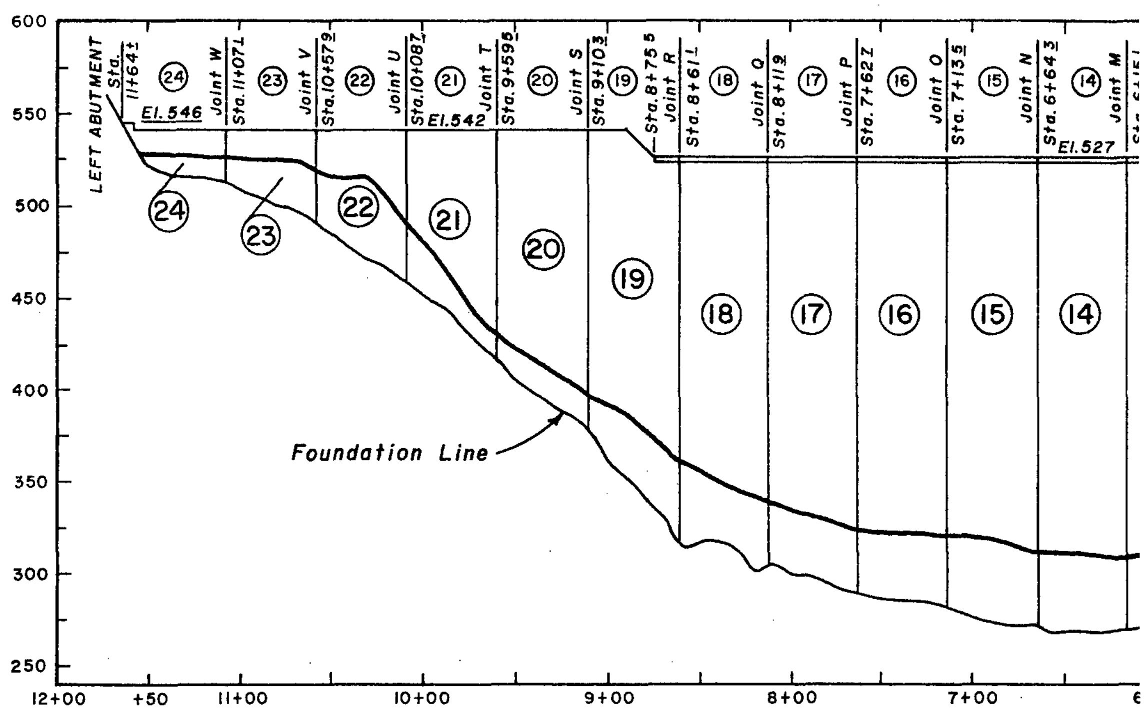
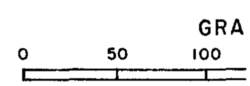
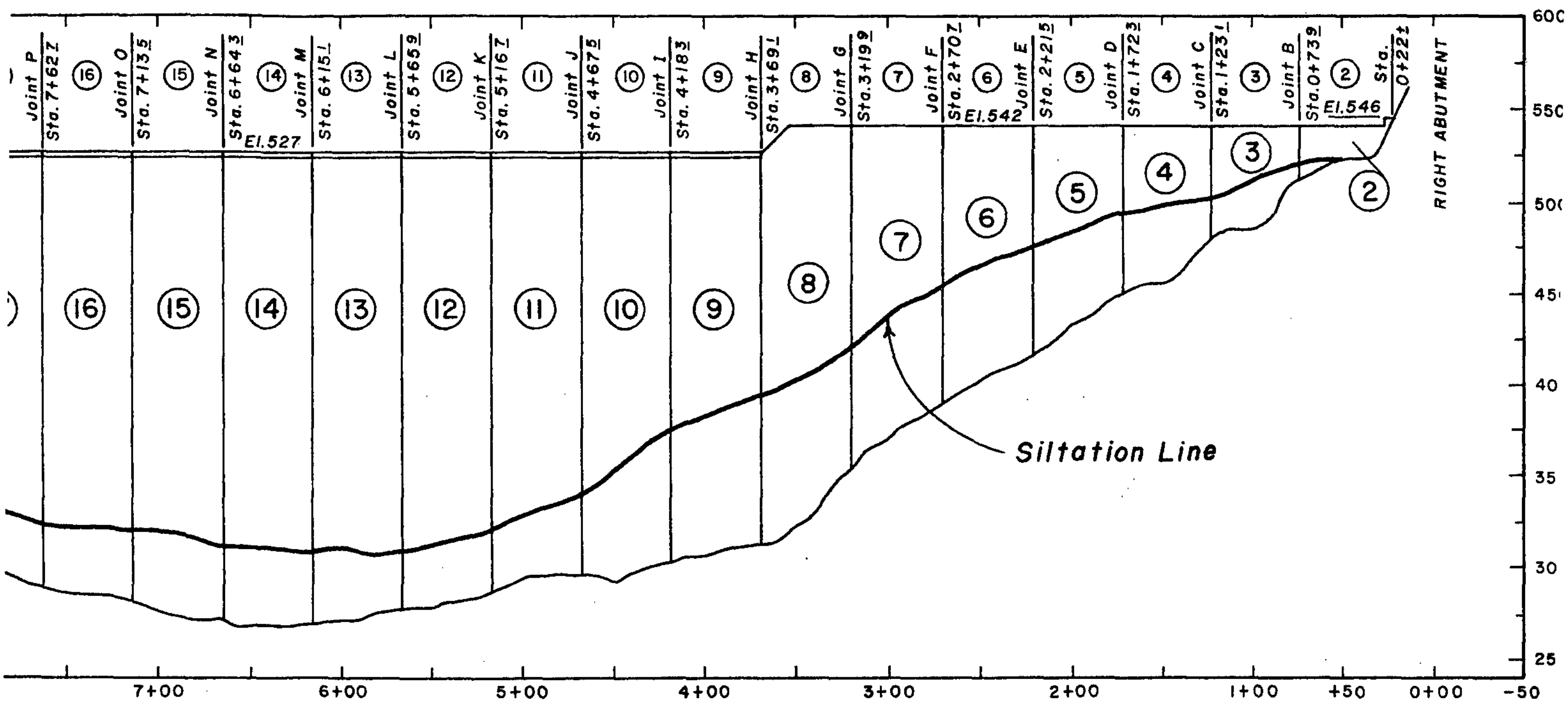


FIGURE 8-1 DEPTH OF SILT AGAINST



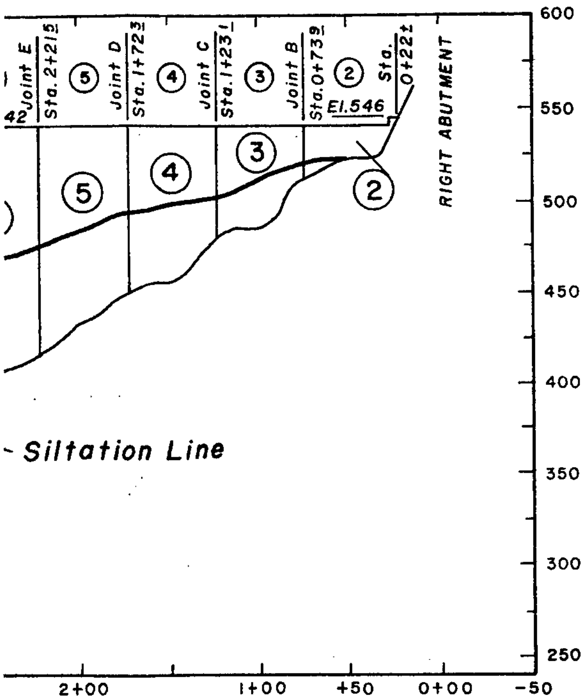


F SILT AGAINST THE UPSTREAM FACE OF ENGLEBRIGHT DAM

GRAPHIC SCALE

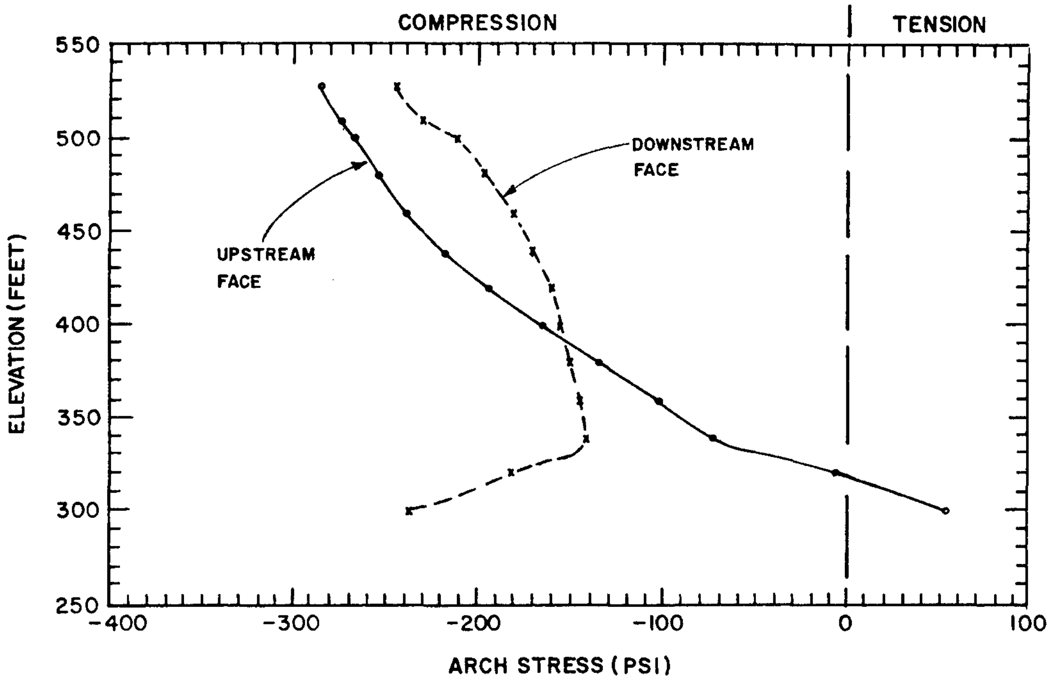
0 50 100 200 300

2


Joint E
Sta. 2+21±
Joint D
Sta. 1+72±
Joint C
Sta. 1+23±
Joint B
Sta. 0+73±
Sta.
0+22±
El. 546
RIGHT ABUTMENT
Siltation Line
600
550
500
450
400
350
300
250
2+00
1+00
+50
0+00
-50

RIGHT DAM

3



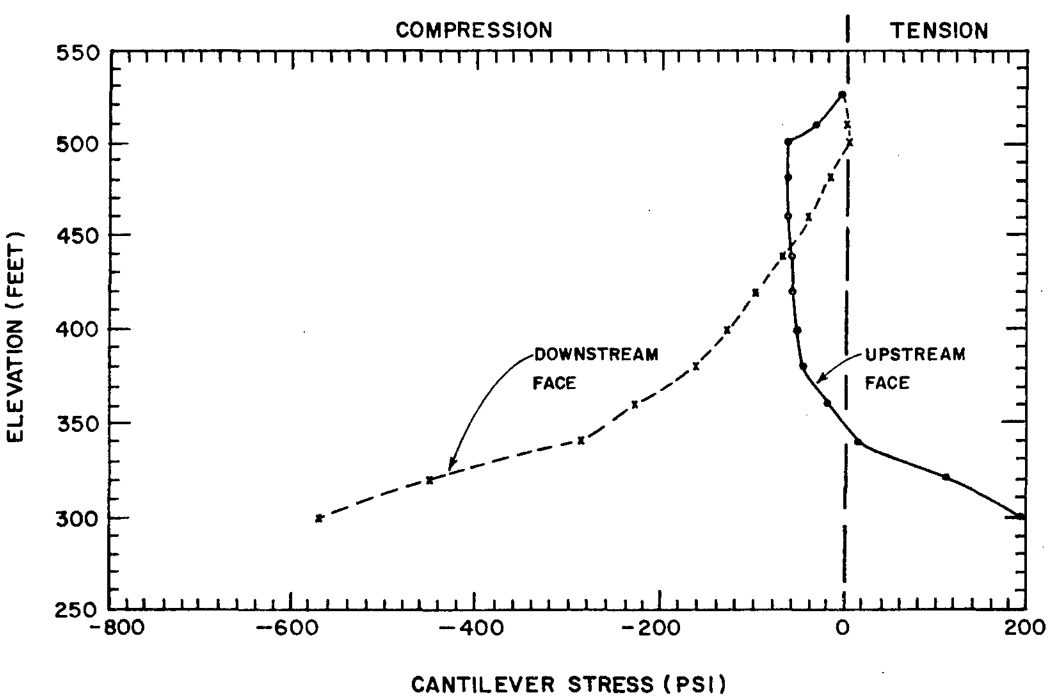
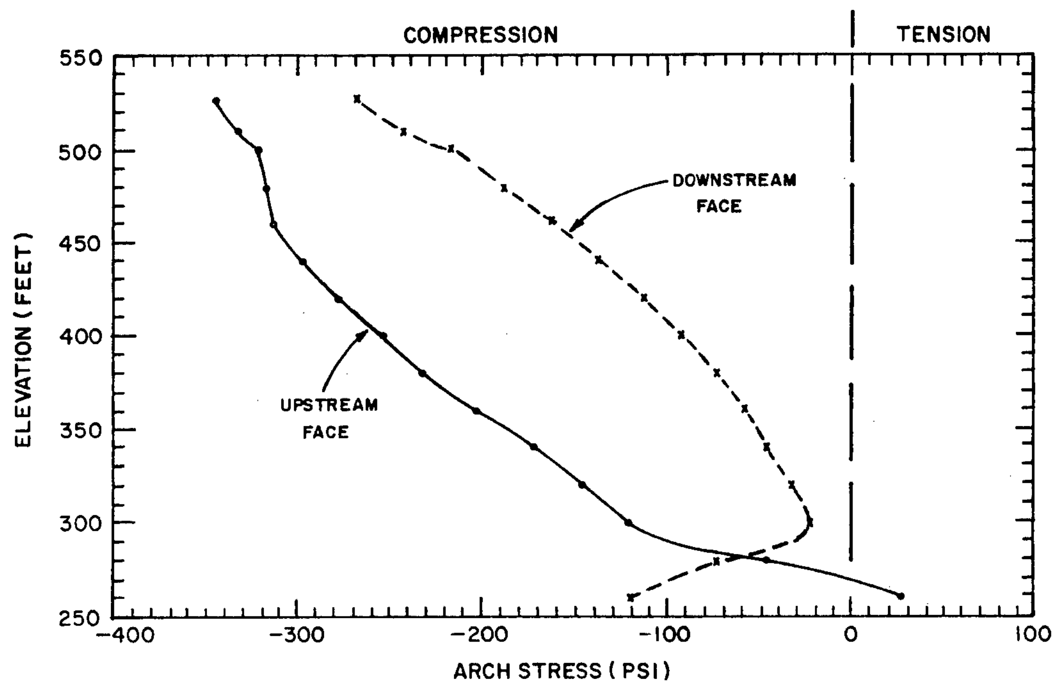


FIGURE 8-2 STATIC ARCH AND CANTILEVER STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG SECTION 6 DUE TO USUAL LOADING. STATIC STRESSES ARE COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).



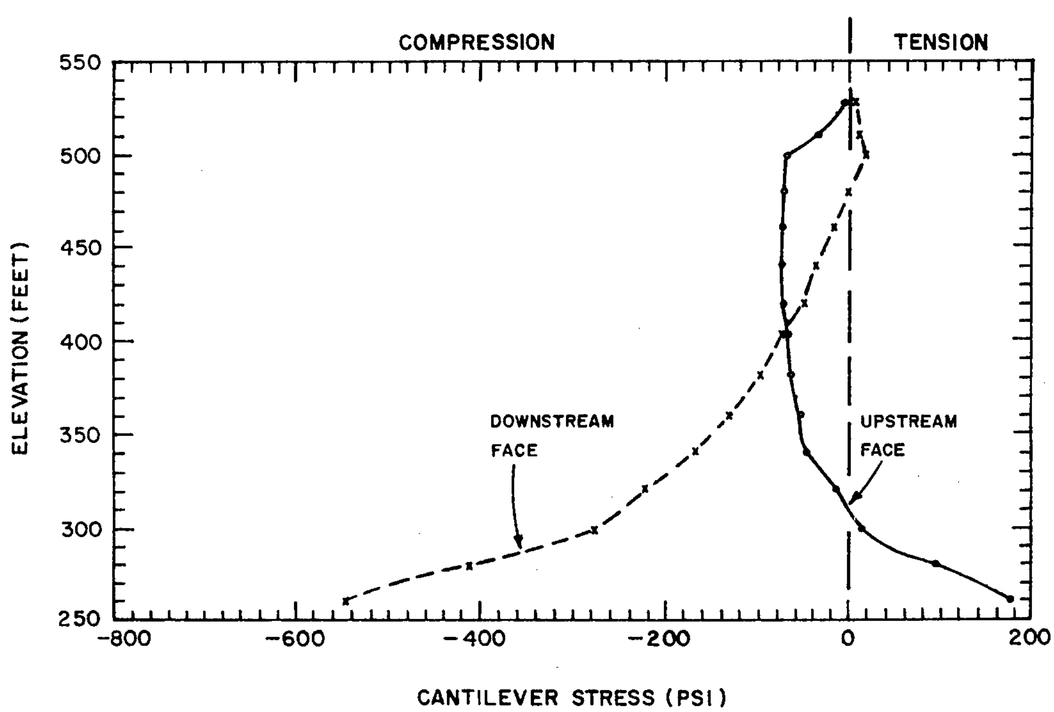


FIGURE 8-3 STATIC ARCH AND CANTILEVER STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG CROWN CANTILEVER DUE TO USUAL LOADING. STATIC STRESSES ARE COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

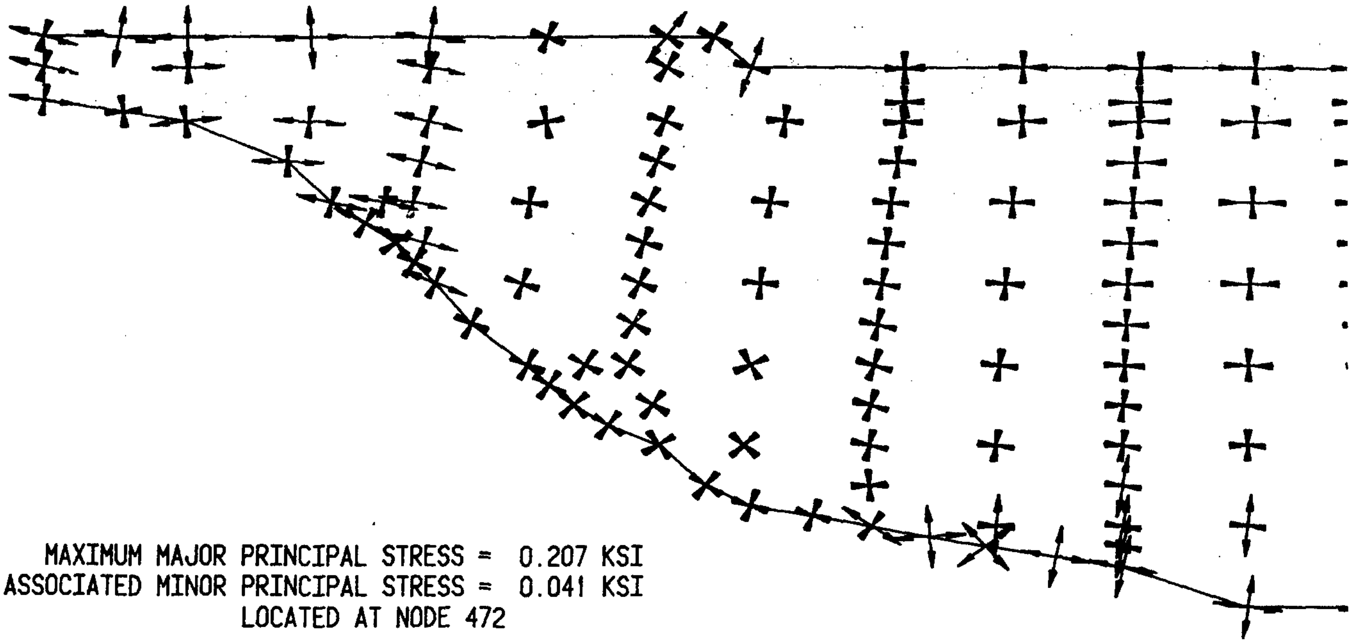


FIGURE 8-4 PLOT OF PRINCIPAL S
ON FLEXIBLE FOUNDATION

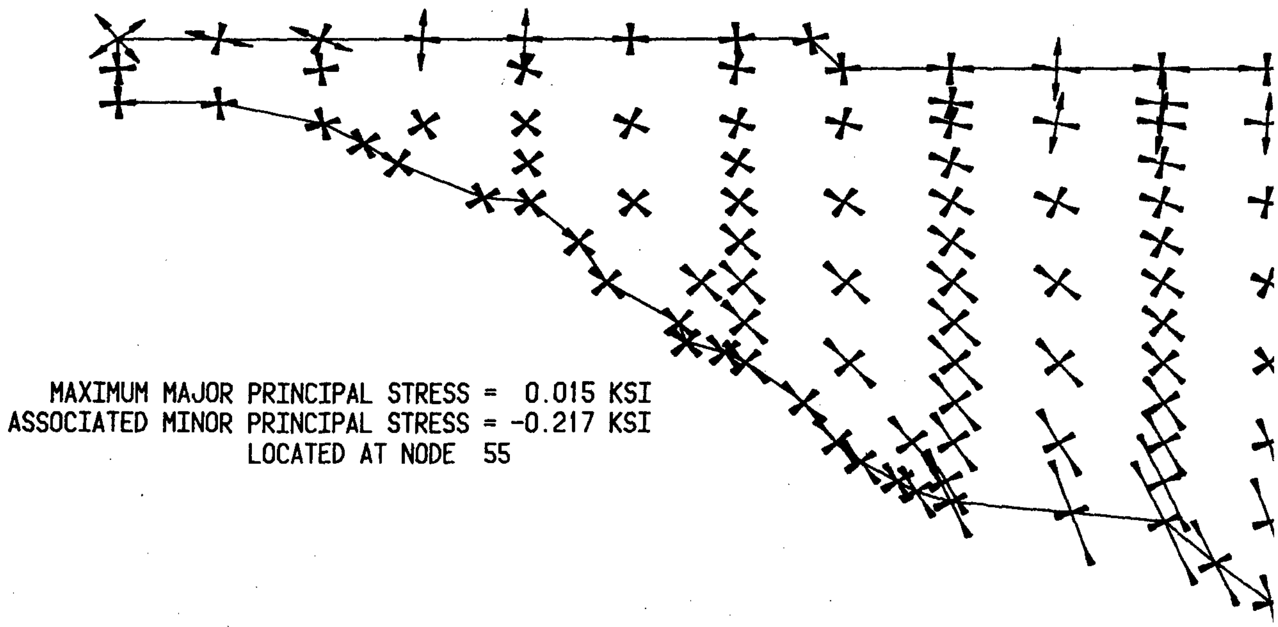


FIGURE 8-5 PLOT OF PRINCIPAL ST
ON FLEXIBLE FOUNDATION

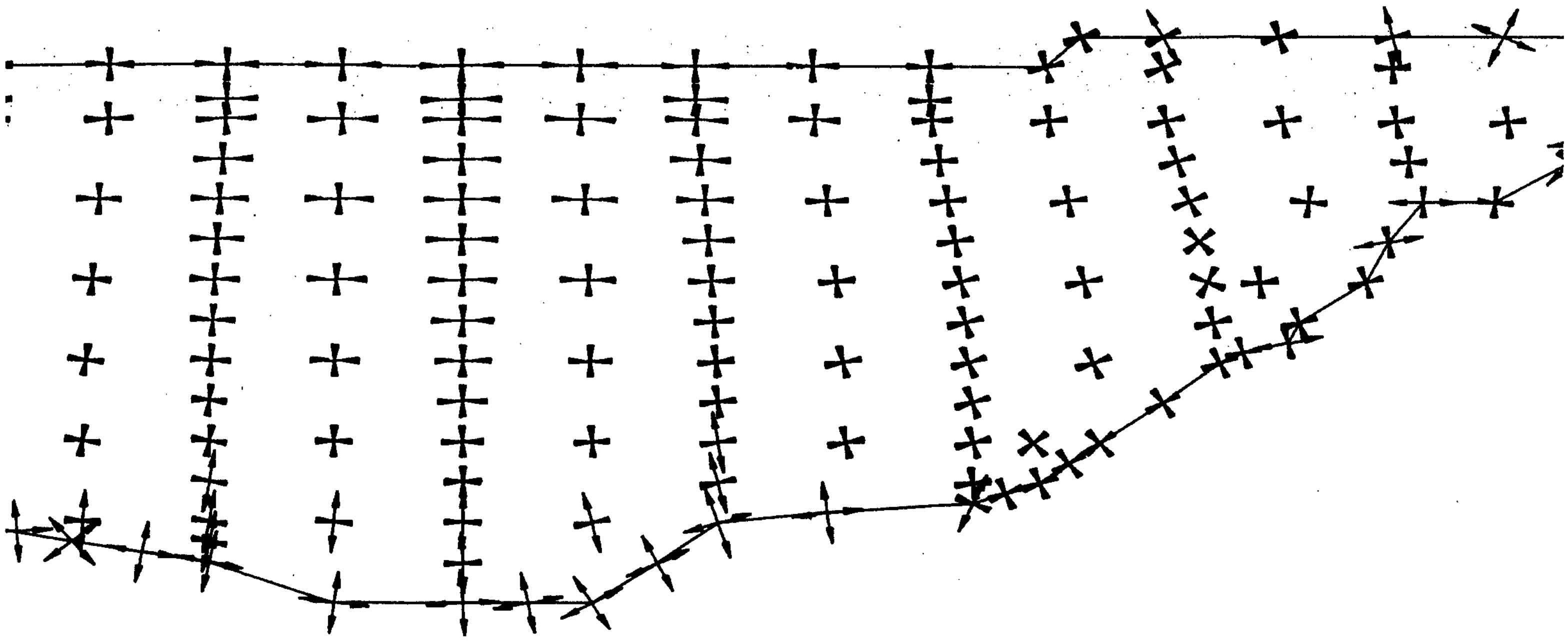


FIGURE 8-4 PLOT OF PRINCIPAL STRESSES ON UPSTREAM FACE OF THE DAM
ON FLEXIBLE FOUNDATION ROCK DUE TO USUAL LOADING

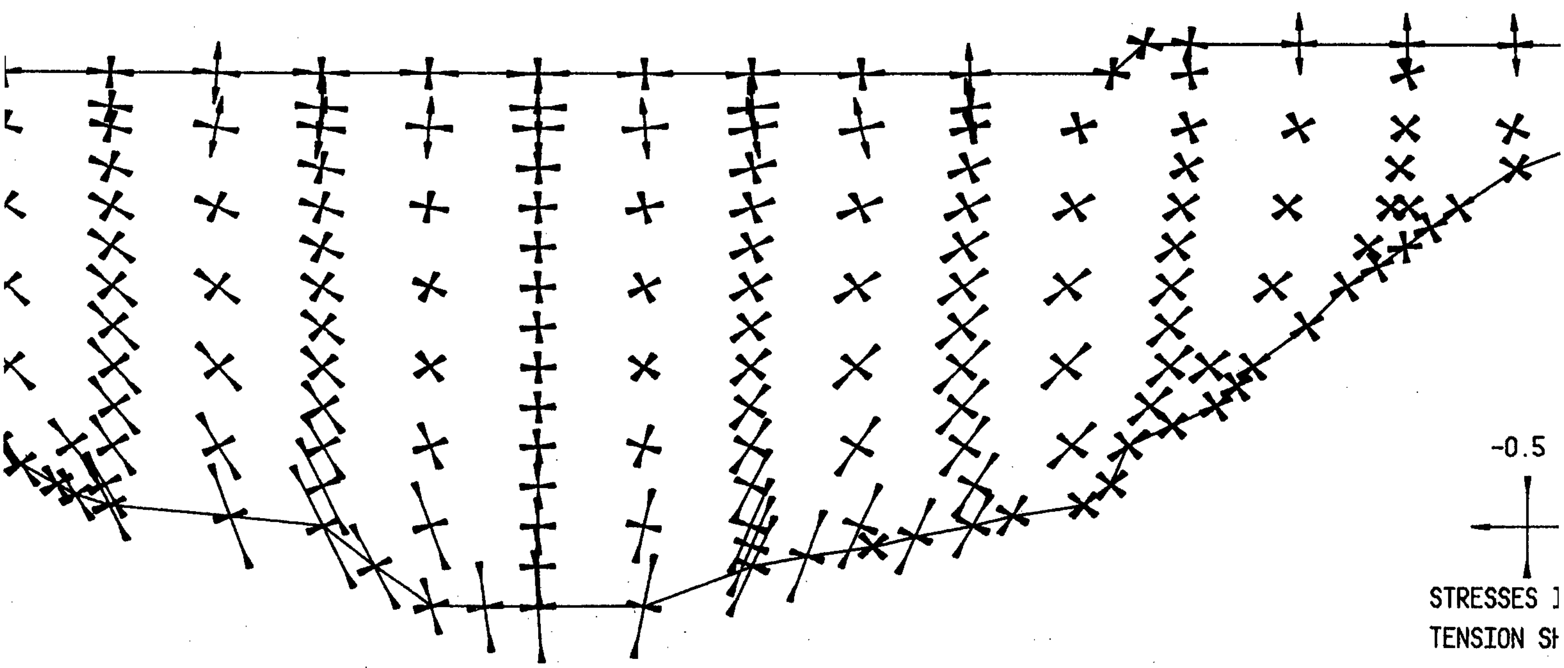
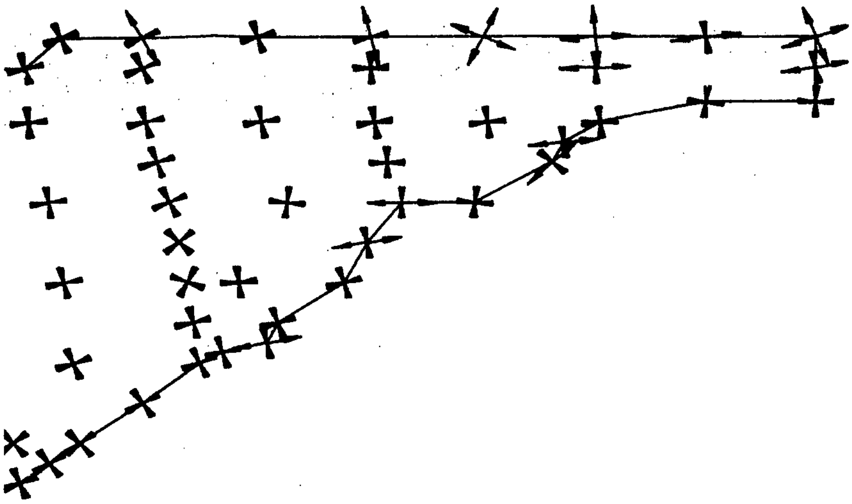
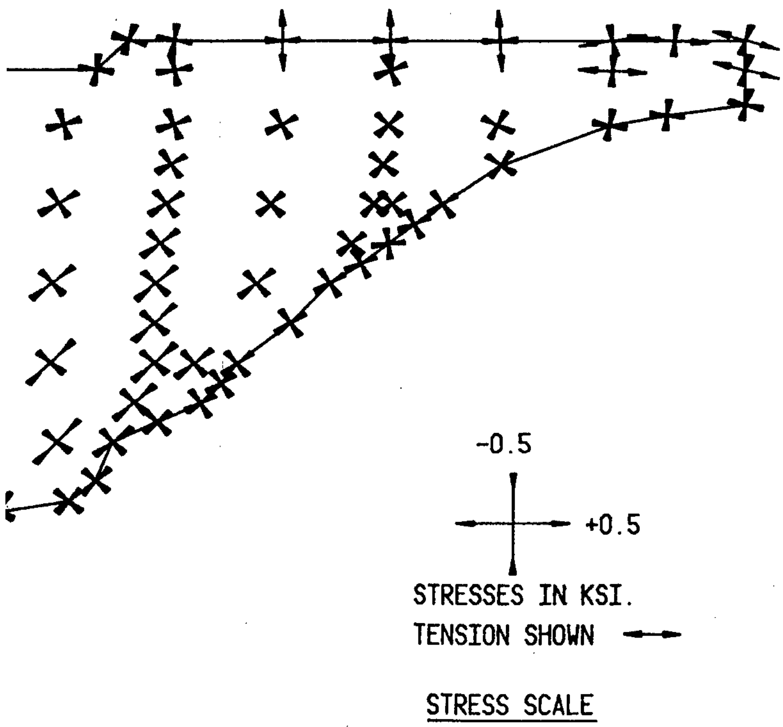


FIGURE 8-5 PLOT OF PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE DAM
ON FLEXIBLE FOUNDATION ROCK DUE TO USUAL LOADING





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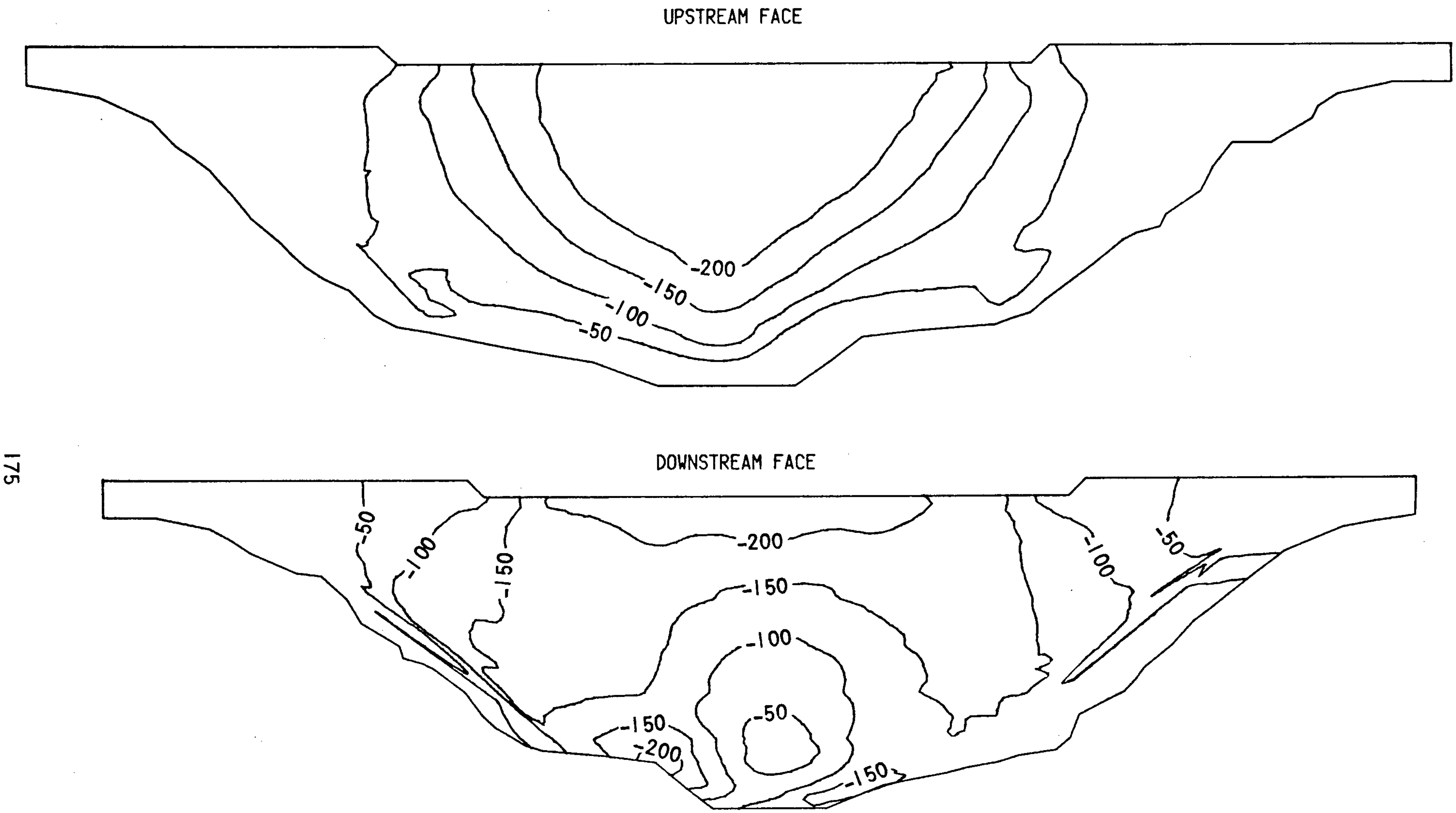


FIGURE 8-6 CONTOUR PLOT OF ARCH STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO USUAL LOADING.

NOTE: STRESSES ARE IN P.S.I.

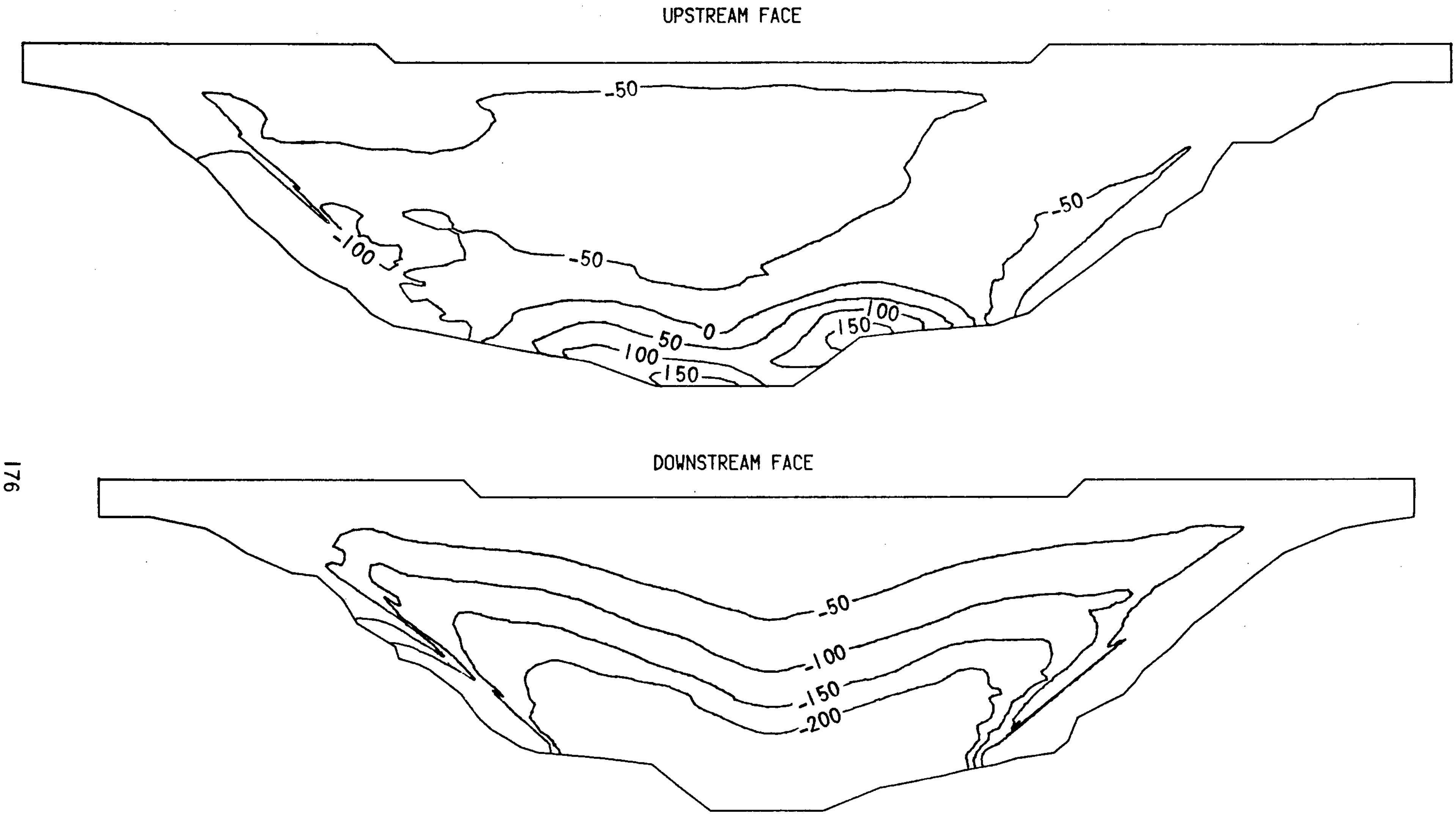


FIGURE 8-7 CONTOUR PLOT OF CANTILEVER STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO USUAL LOADING.

NOTE: STRESSES ARE IN P.S.I.

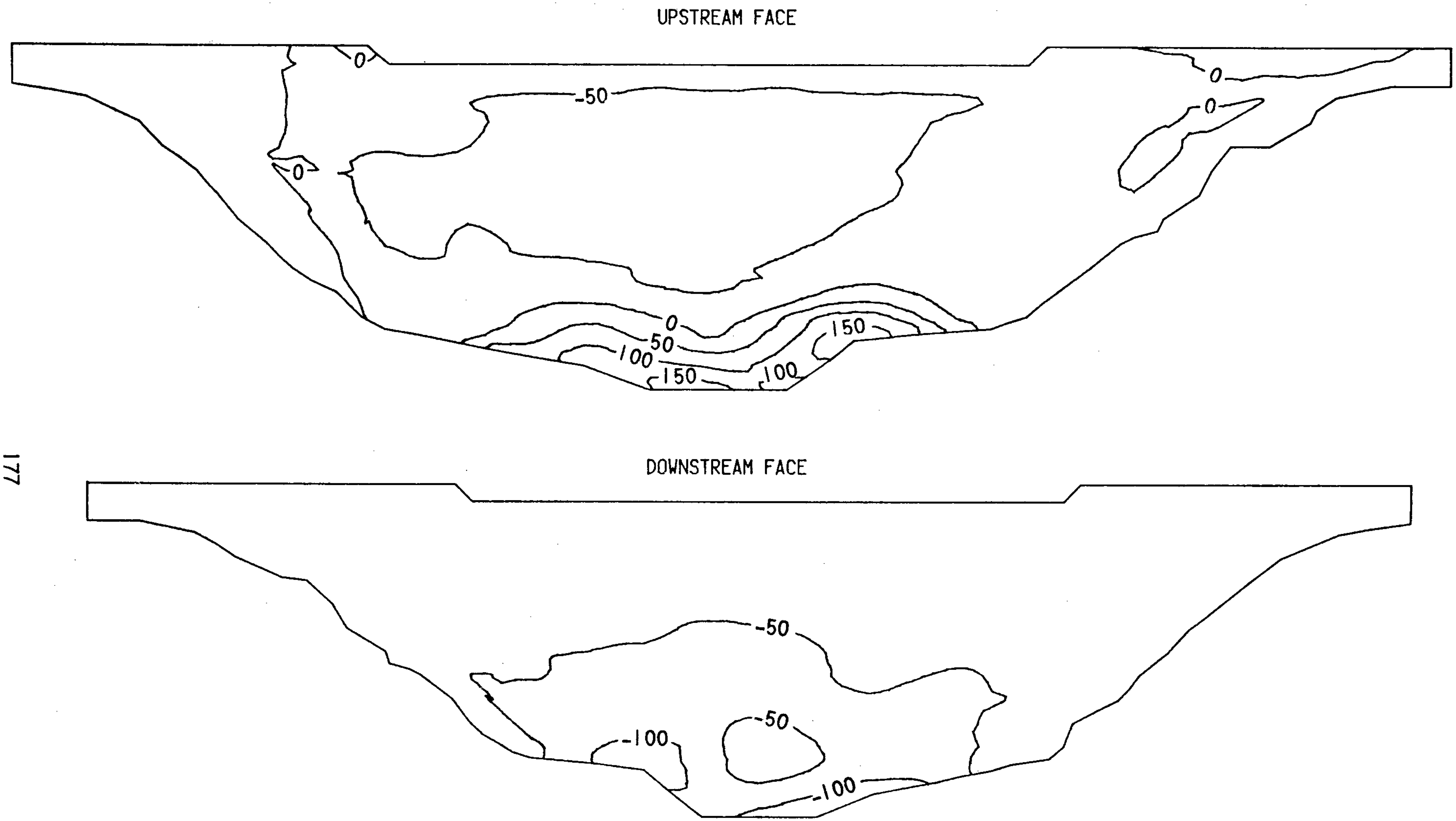


FIGURE 8-8 CONTOUR PLOT OF MAJOR PRINCIPAL STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO USUAL LOADING.
NOTE: STRESSES ARE IN P.S.I.

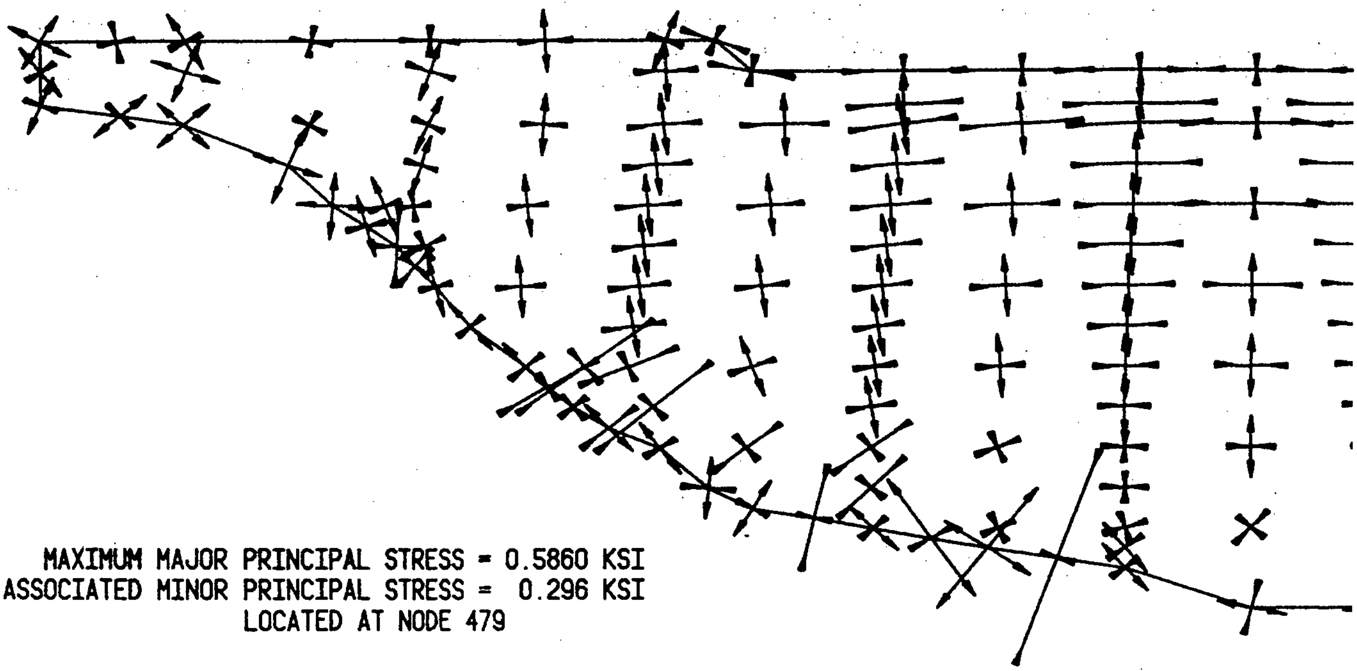


FIGURE 8-9 PLOT OF PRINCIPAL S
ON FLEXIBLE FOUNDATION

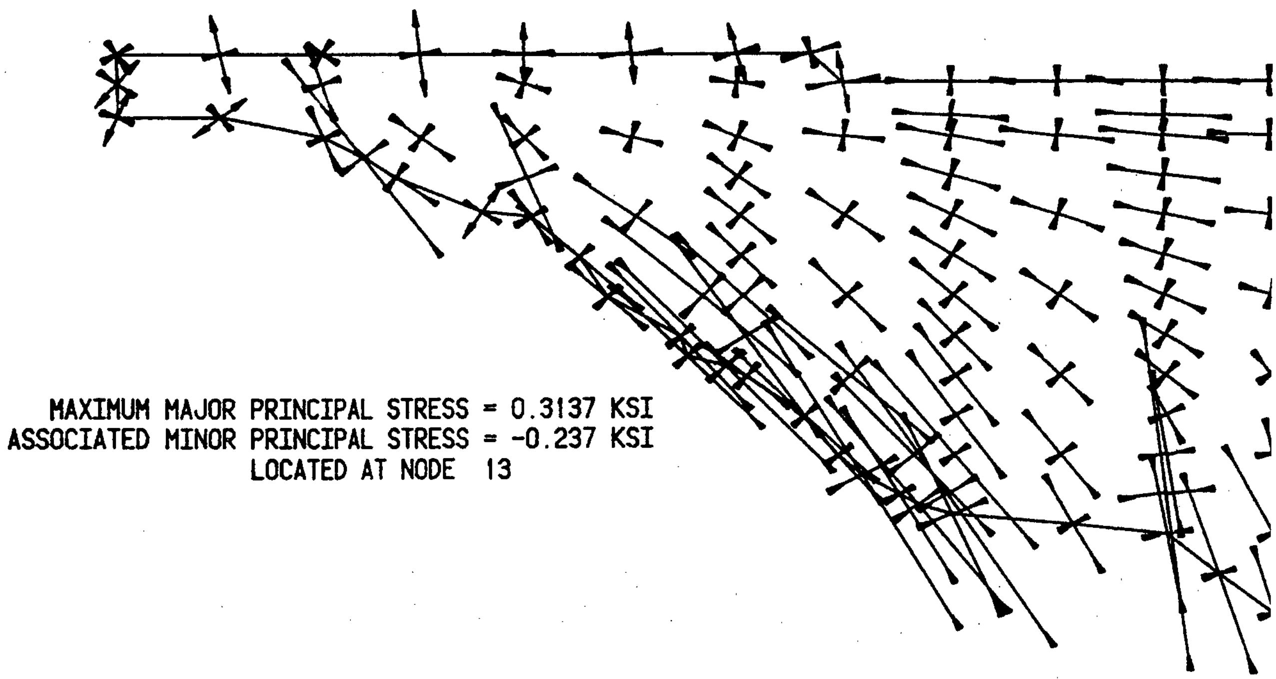
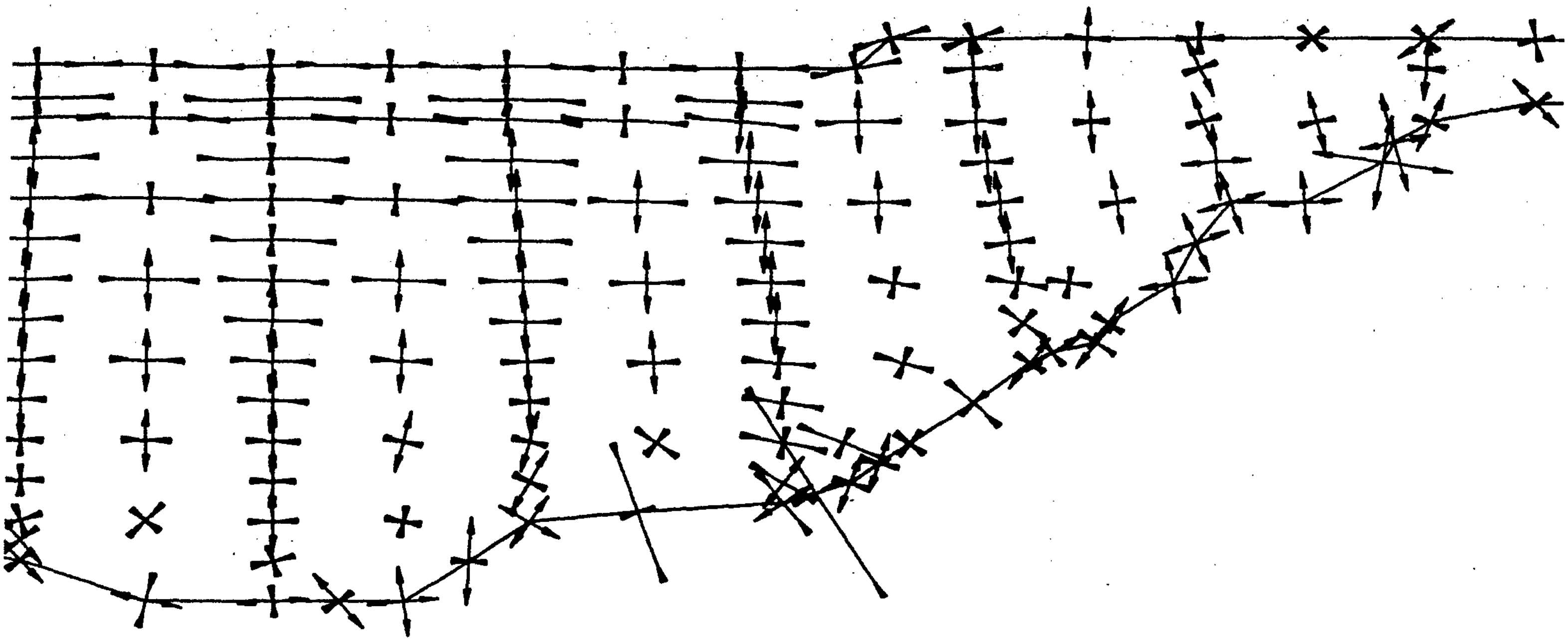
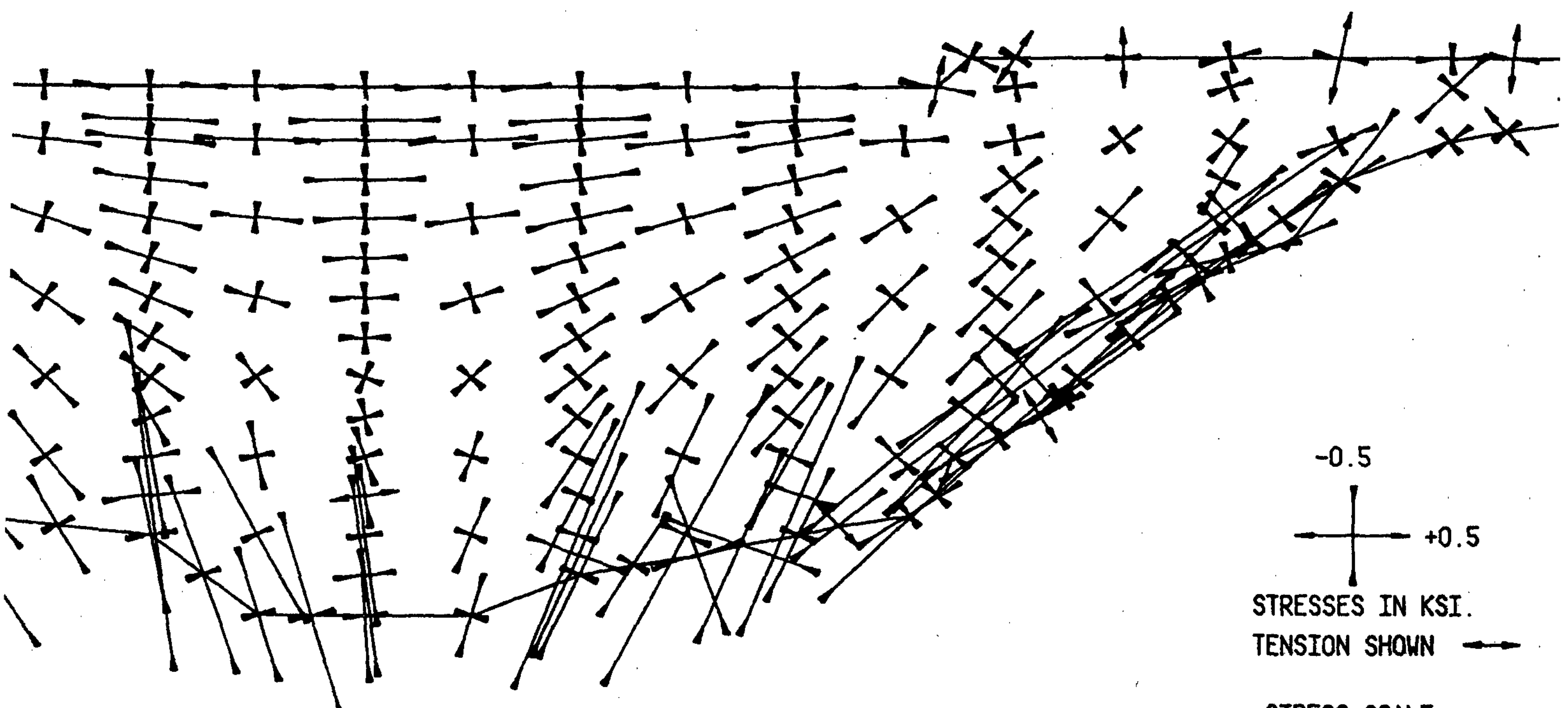


FIGURE 8-10 PLOT OF PRINCIPAL S
ON FLEXIBLE FOUNDATION

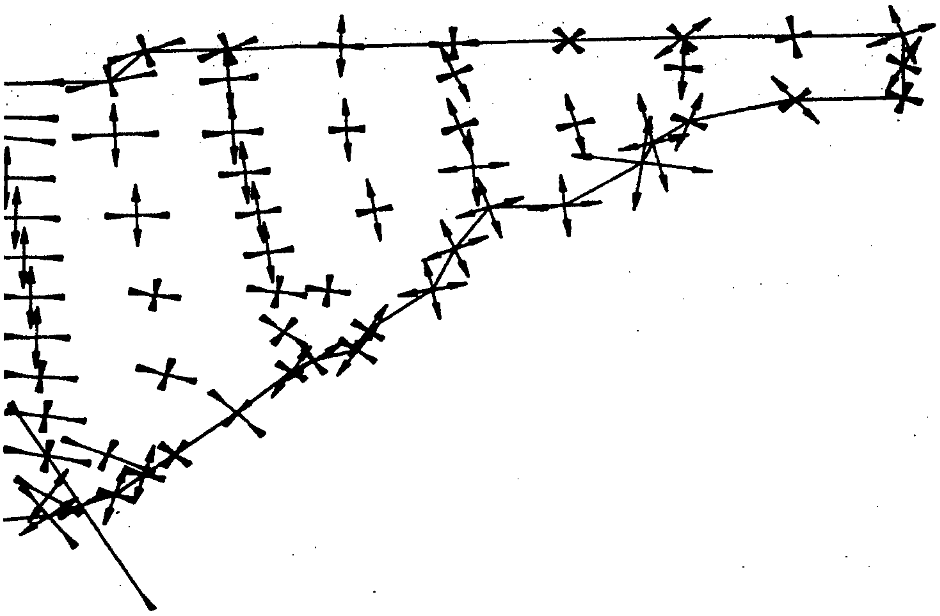


'LOT OF PRINCIPAL STRESSES ON UPSTREAM FACE OF THE DAM
IBLE FOUNDATION ROCK DUE TO SURCHARGE LOADING



'LOT OF PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE DAM
KIBLE FOUNDATION ROCK DUE TO SURCHARGE LOADING

2



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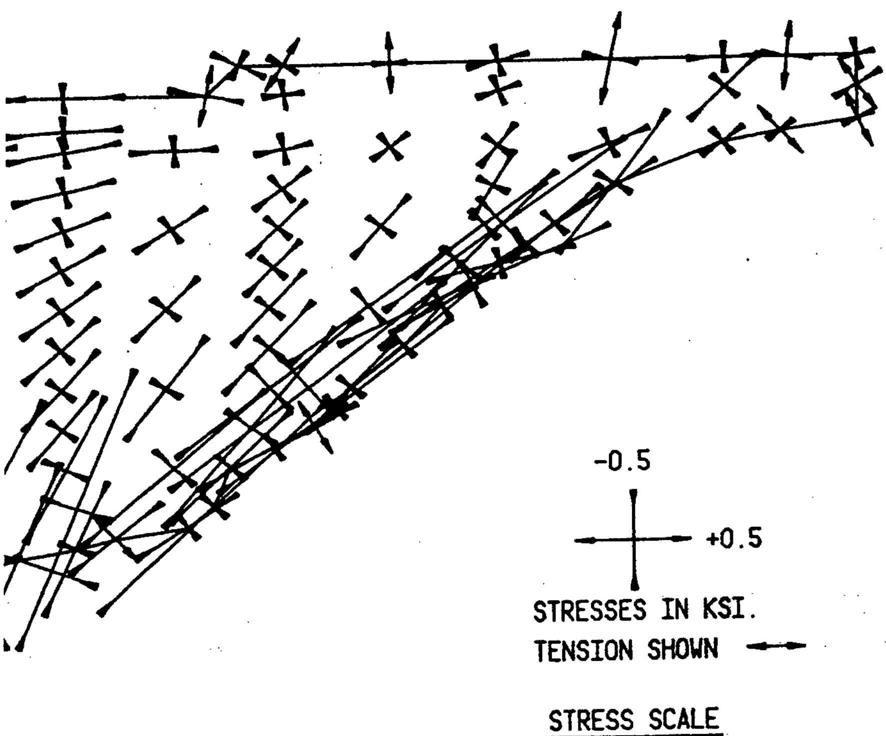


DAM

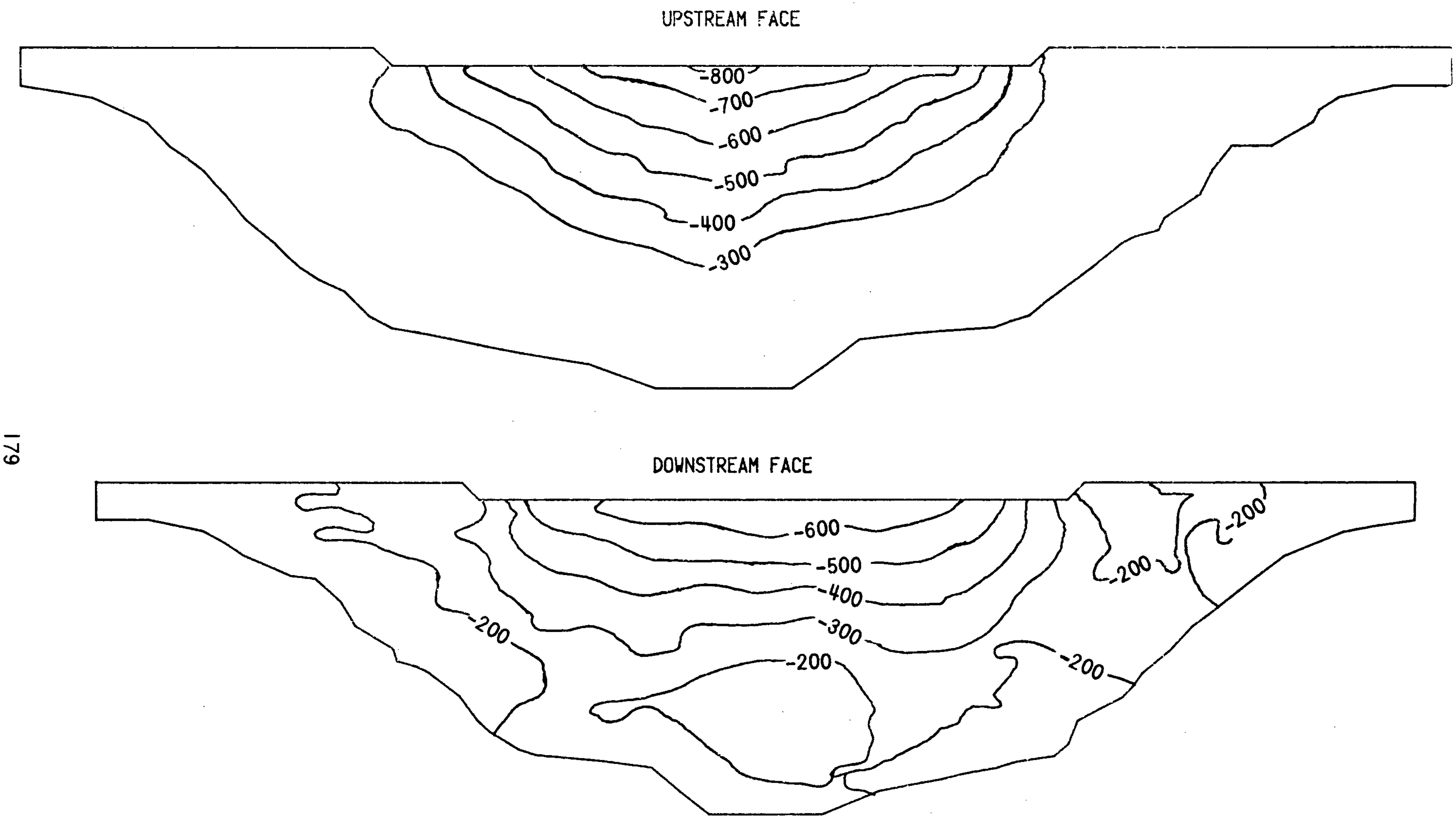


FIGURE 8-11 CONTOUR PLOT OF ARCH STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO SURCHARGE LOADING.
NOTE: ALL STRESSES ARE IN PSI.

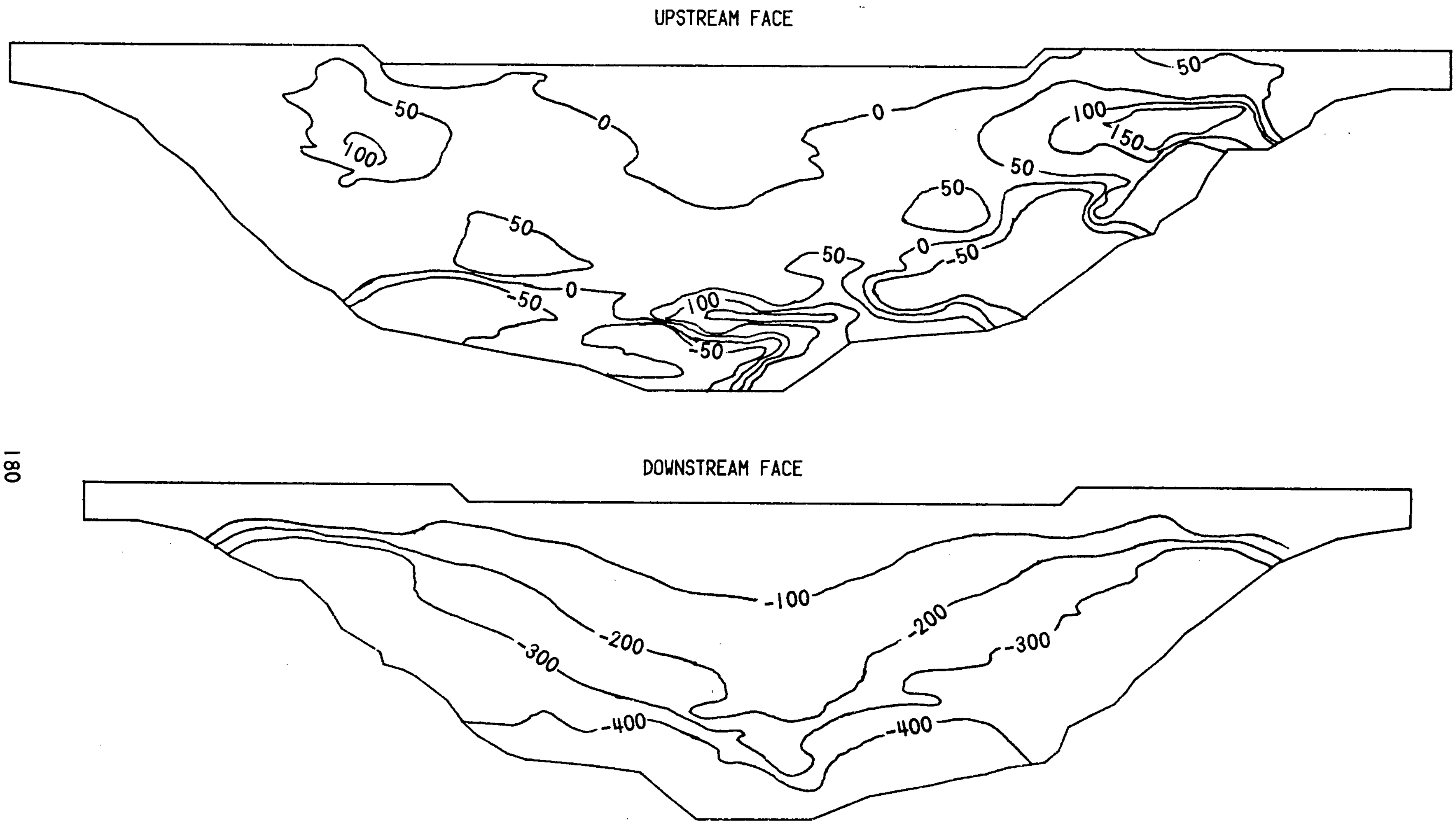


FIGURE 8-12 CONTOUR PLOT OF CANTILEVER STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO SURCHARGE LOADING.
NOTE: ALL STRESSES ARE IN PSI.

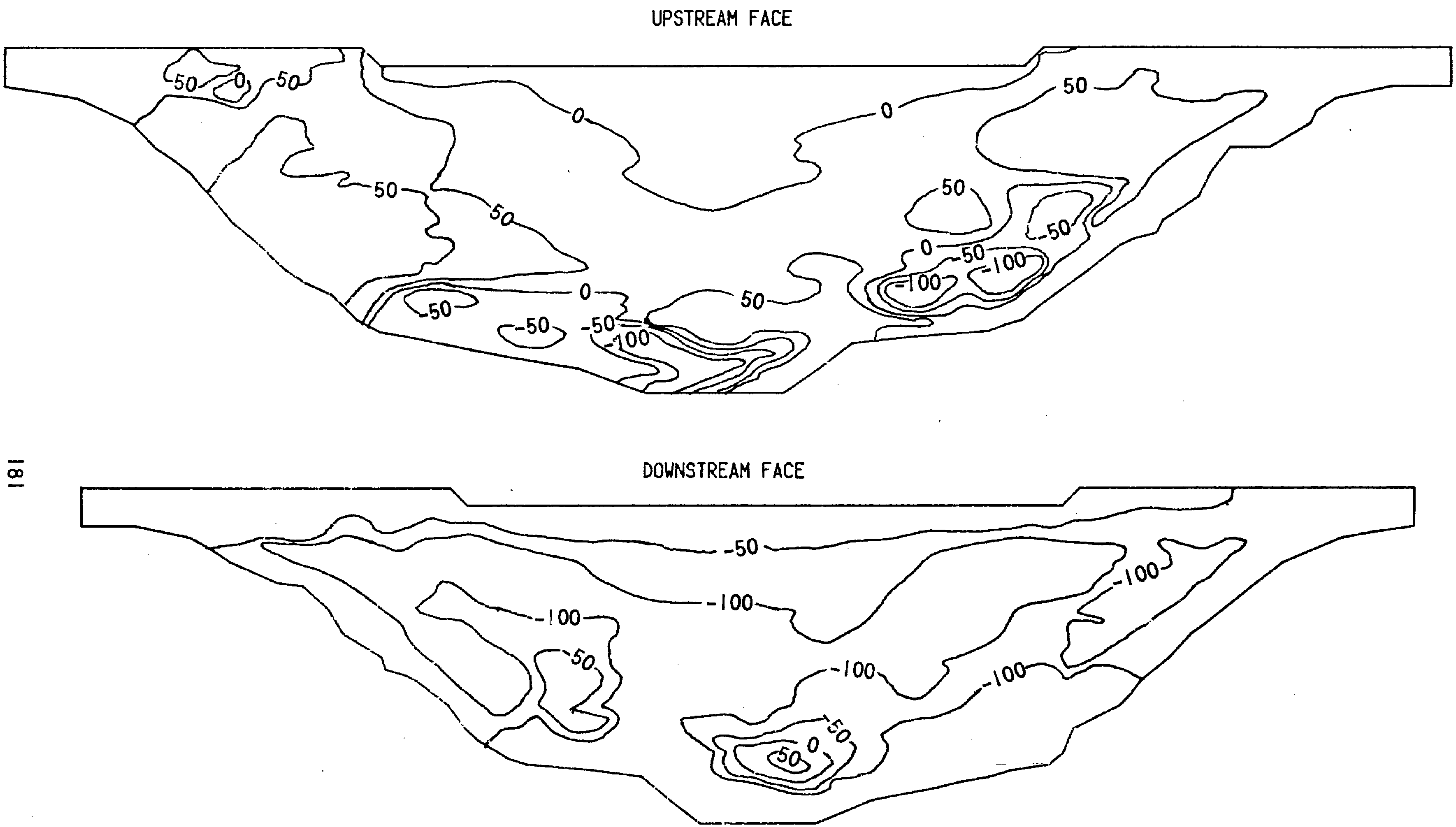


FIGURE 8-13 CONTOUR PLOT OF MAJOR PRINCIPAL STRESSES ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK DUE TO SURCHARGE LOADING.
NOTE: ALL STRESSES ARE IN PSI.

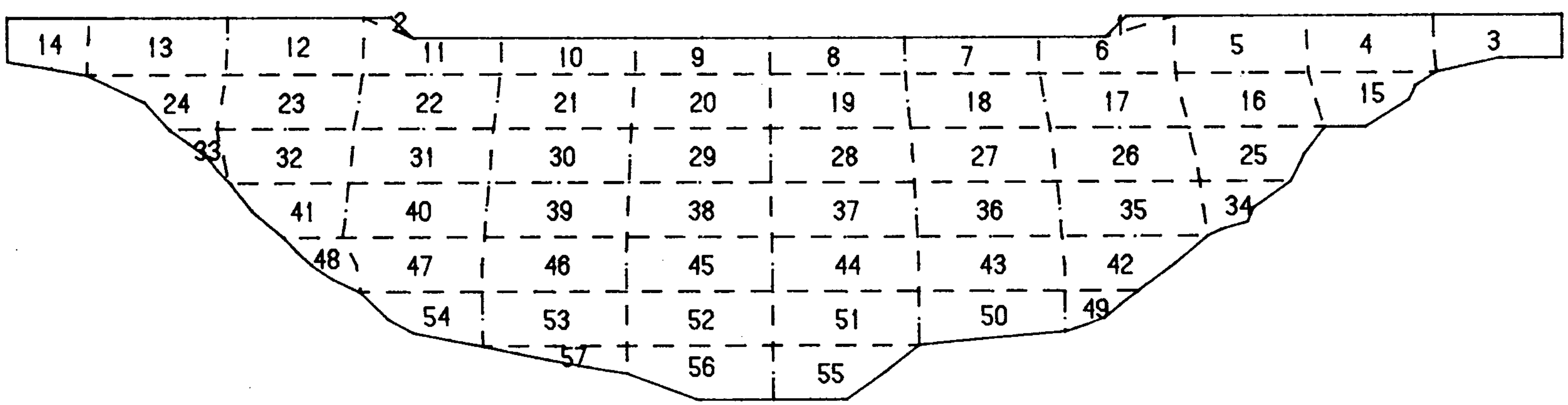


FIGURE 9-1 DEVELOPED VIEW OF UPSTREAM FACE, LOOKING DOWNSTREAM
ELEMENT NUMBERS ARE SHOWN

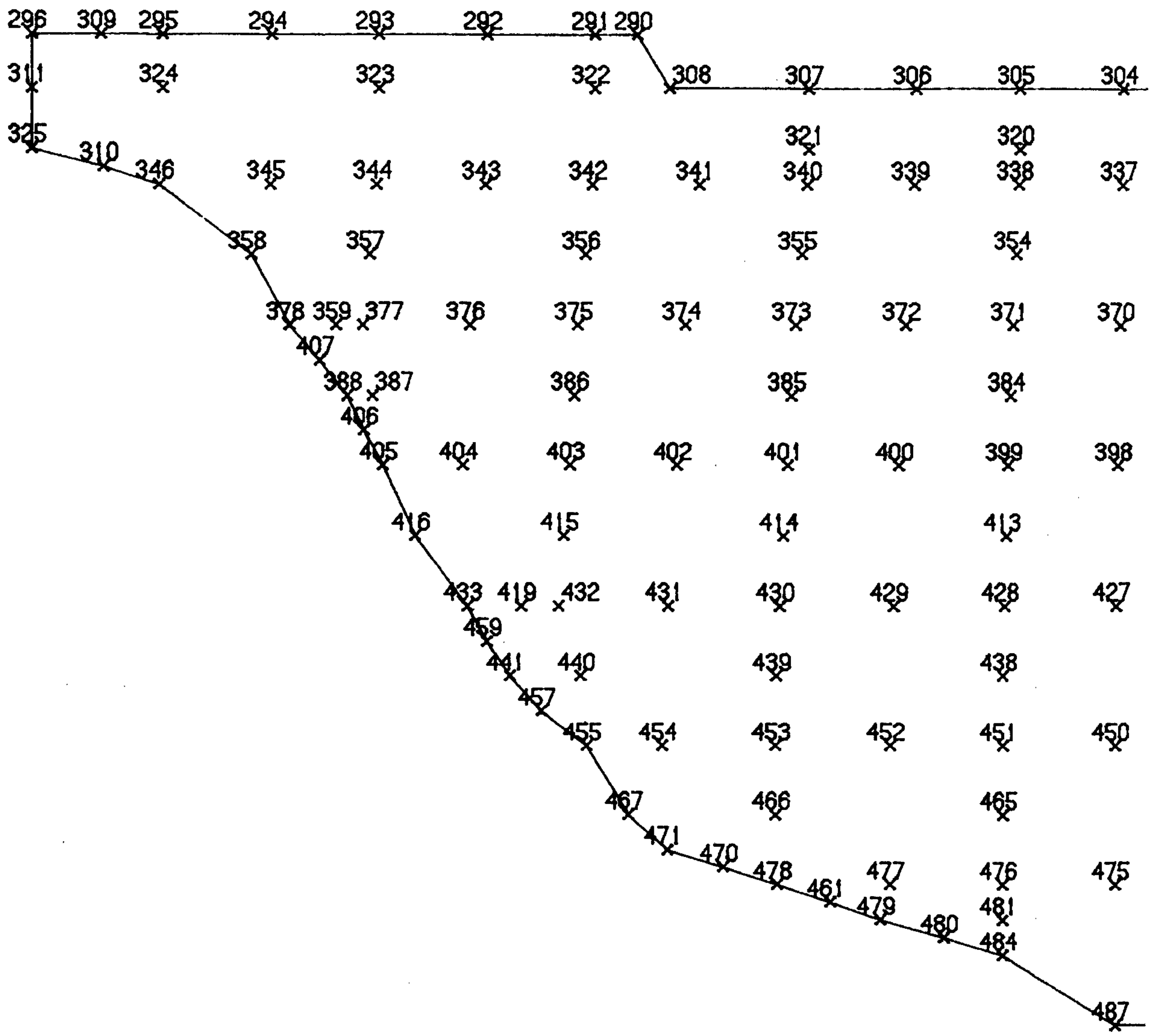
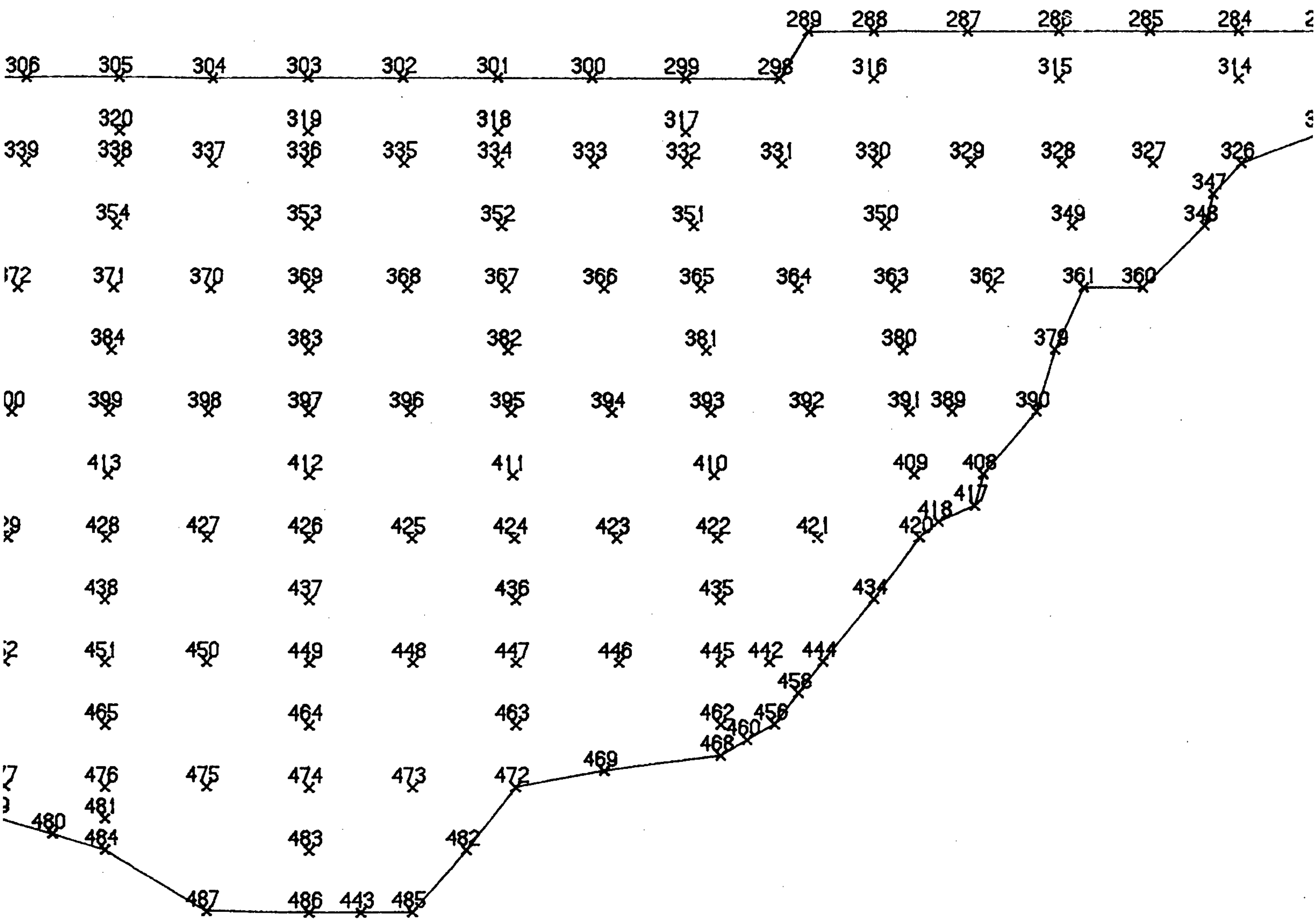


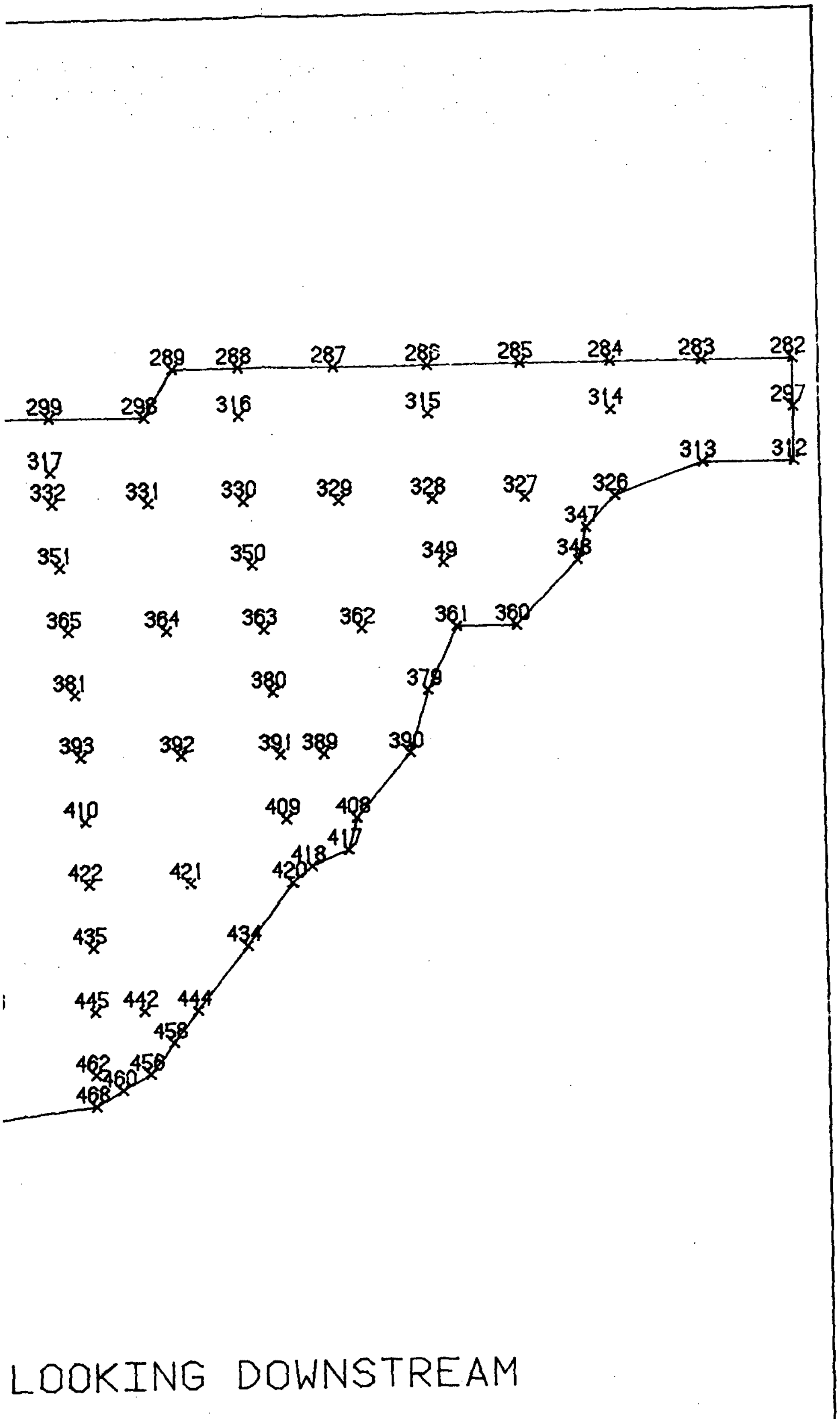
FIGURE 9-2 DEVELOPED VIEW OF

NODE NUMBE



VIEW OF UPSTREAM FACE, LOOKING DOWNSTREAM

DDE NUMBERS ARE SHOWN


289 288 287 286 285 284 283 282
299 298 316 315 314 297
313 312
317
332 331 330 329 328 327 326
347
351 350 349 348
365 364 363 362 361 360
381 380 379
393 392 391 389 390
410 409 408
417
418
422 421 420
435 434
445 442 444
458
462 456
460
468
LOOKING DOWNSTREAM

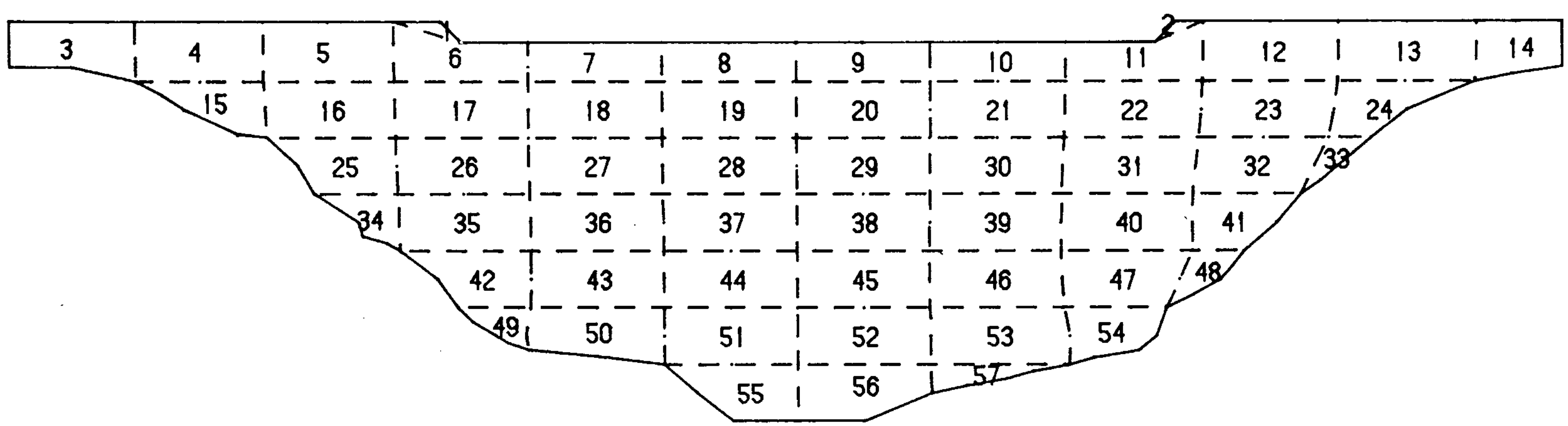


FIGURE 9-3 DEVELOPED VIEW OF DOWNSTREAM FACE, LOOKING UPSTREAM
ELEMENT NUMBERS ARE SHOWN

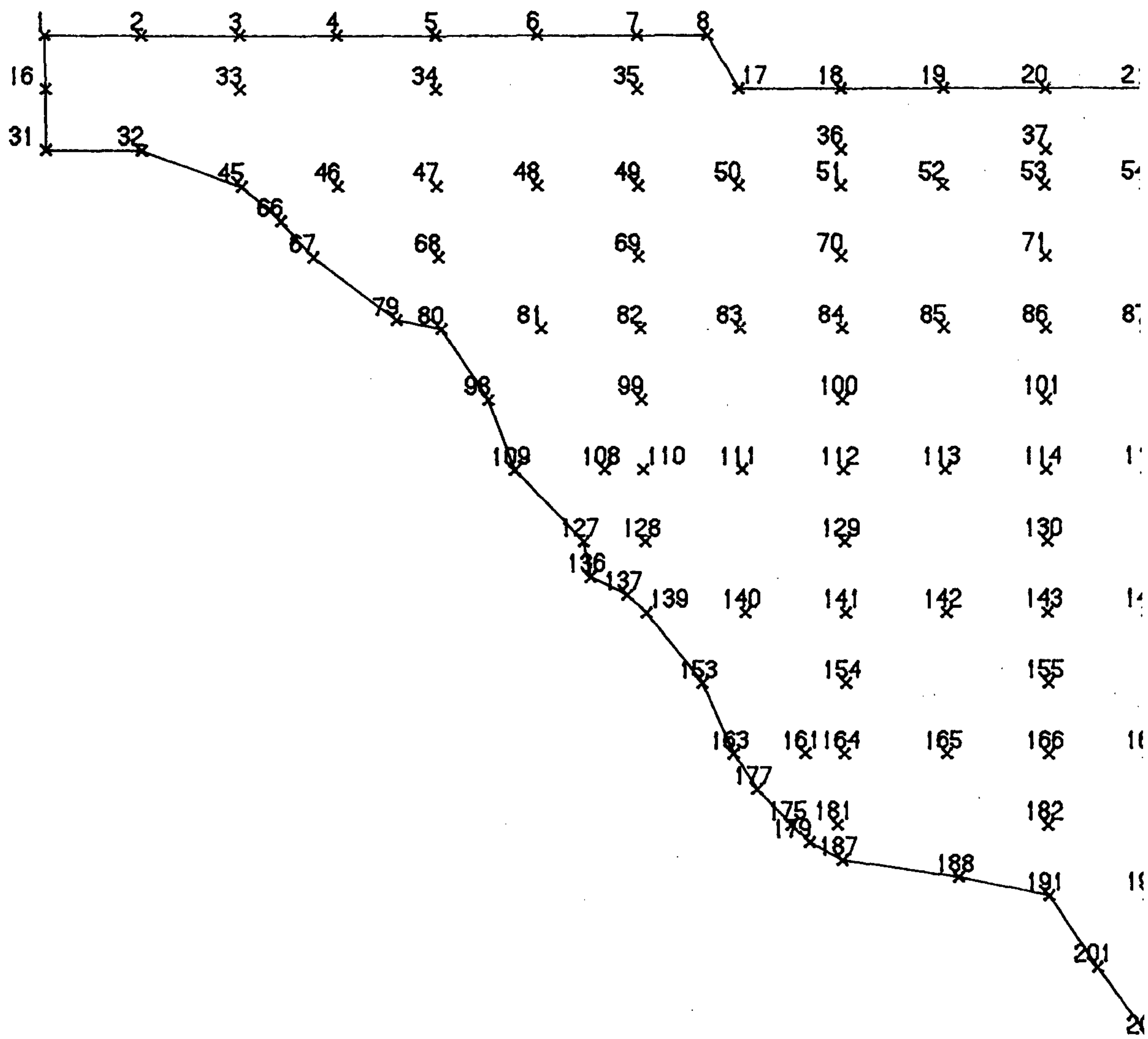
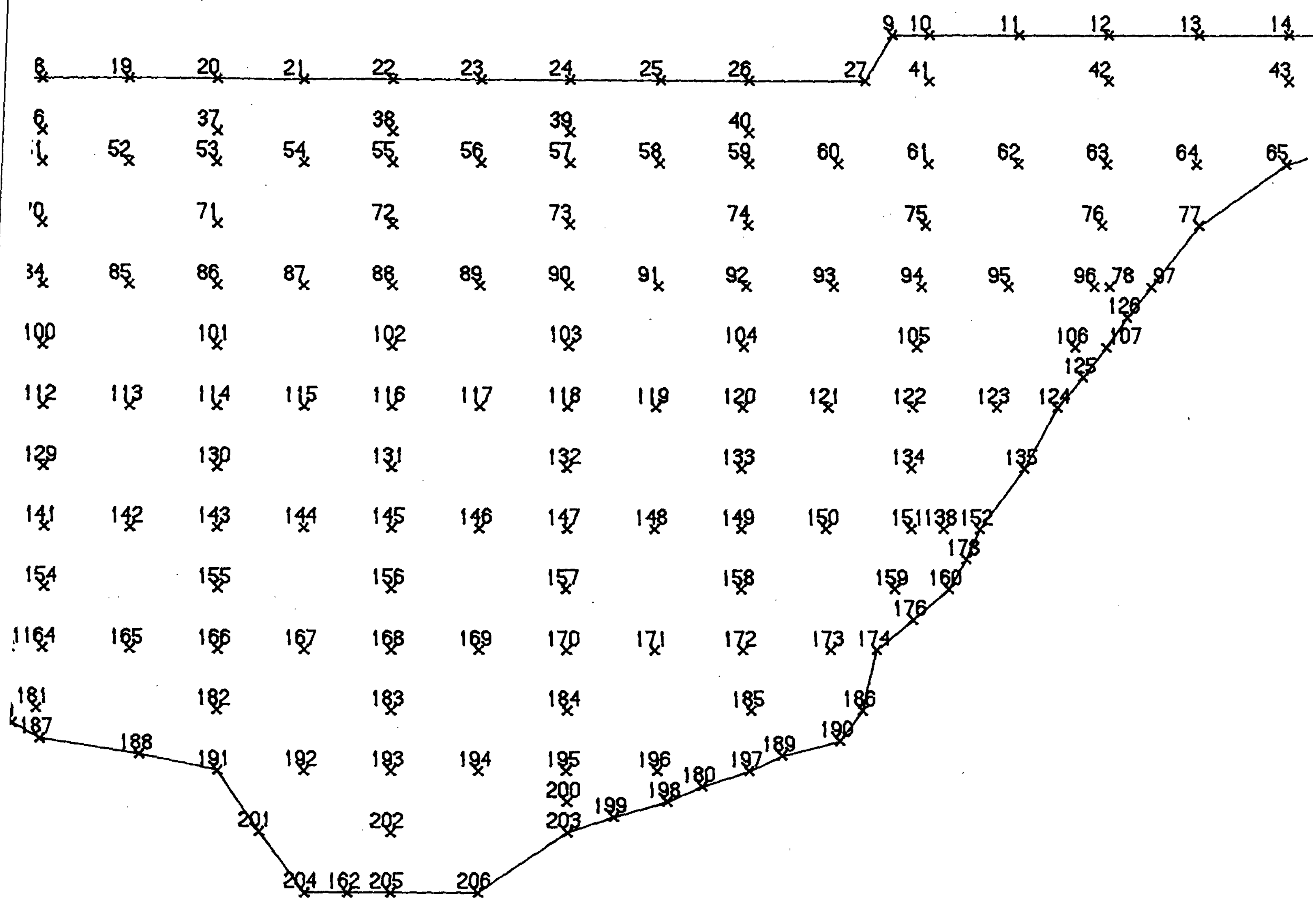


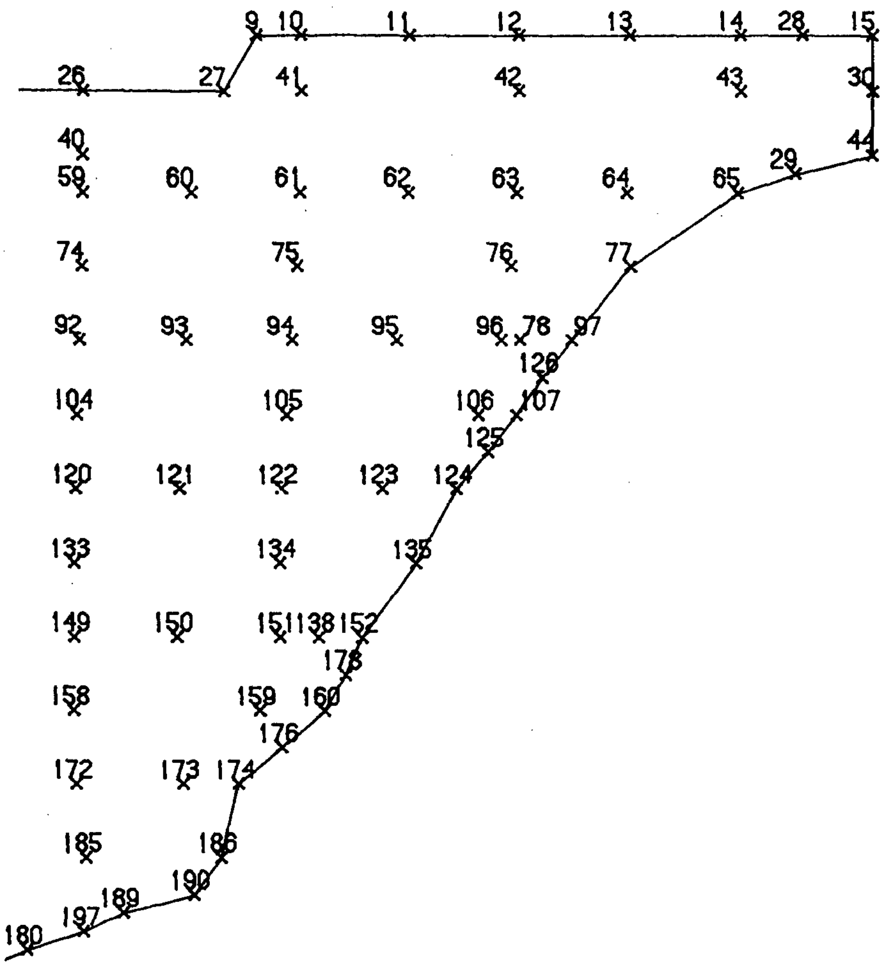
FIGURE 9-4 DEVELOPED VIEW OF

NODE NUMBE



'ED VIEW OF DOWNSTREAM FACE, LOOKING UPSTREAM

NODE NUMBERS ARE SHOWN



:E, LOOKING UPSTREAM

STRESSES COMPUTED AT GAUSS QUADRATURE POINTS

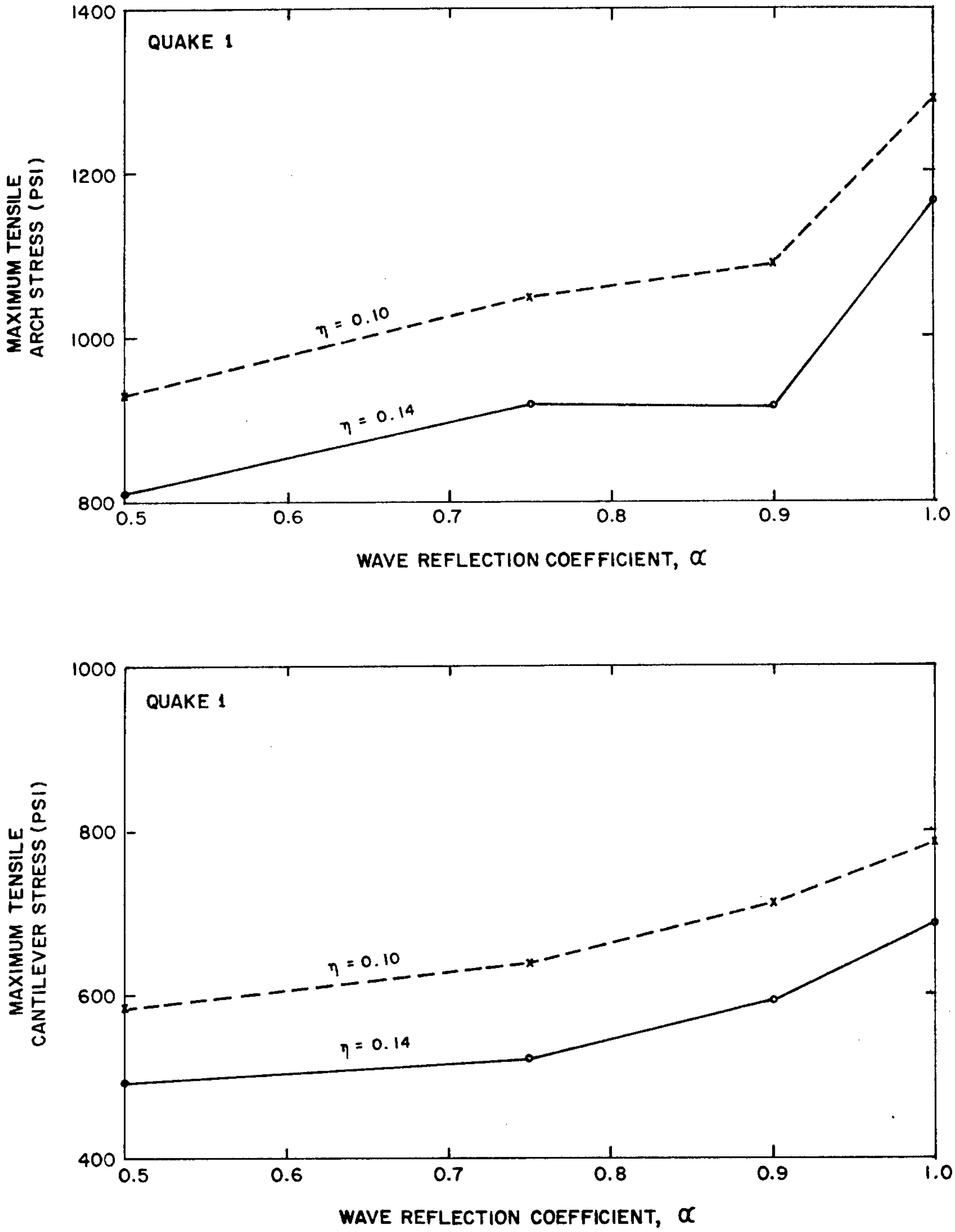
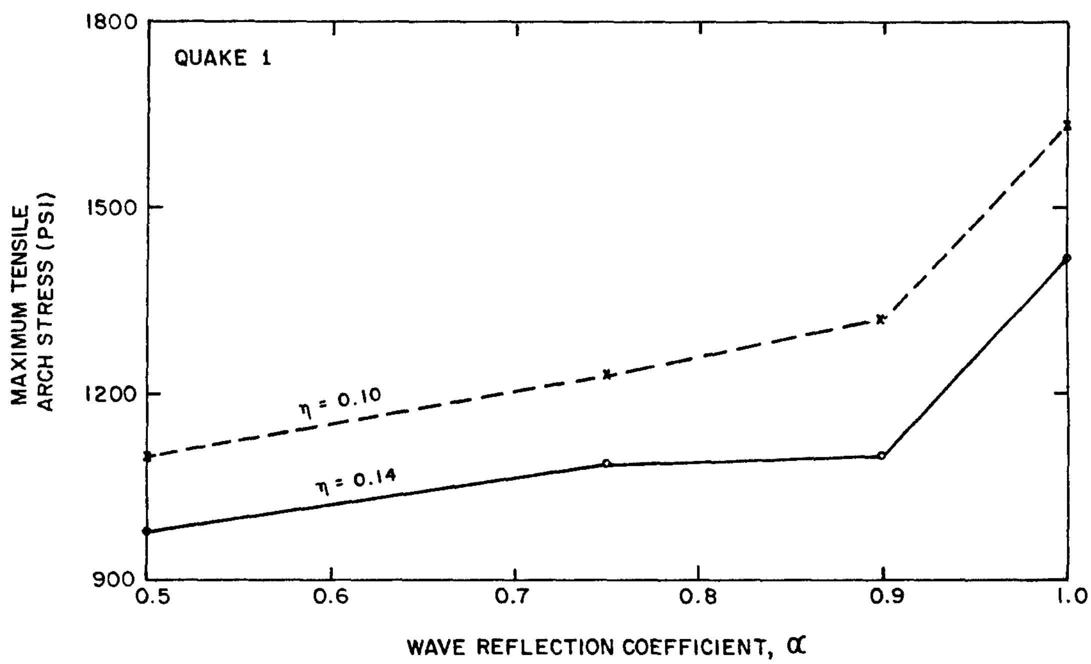


FIGURE 9-5 MAXIMUM TENSILE STRESSES (DUE TO QUAKE I EXCITATION) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) VERSUS α FOR η = 0.10 AND η = 0.14. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

STRESSES COMPUTED AT DAM NODAL POINTS



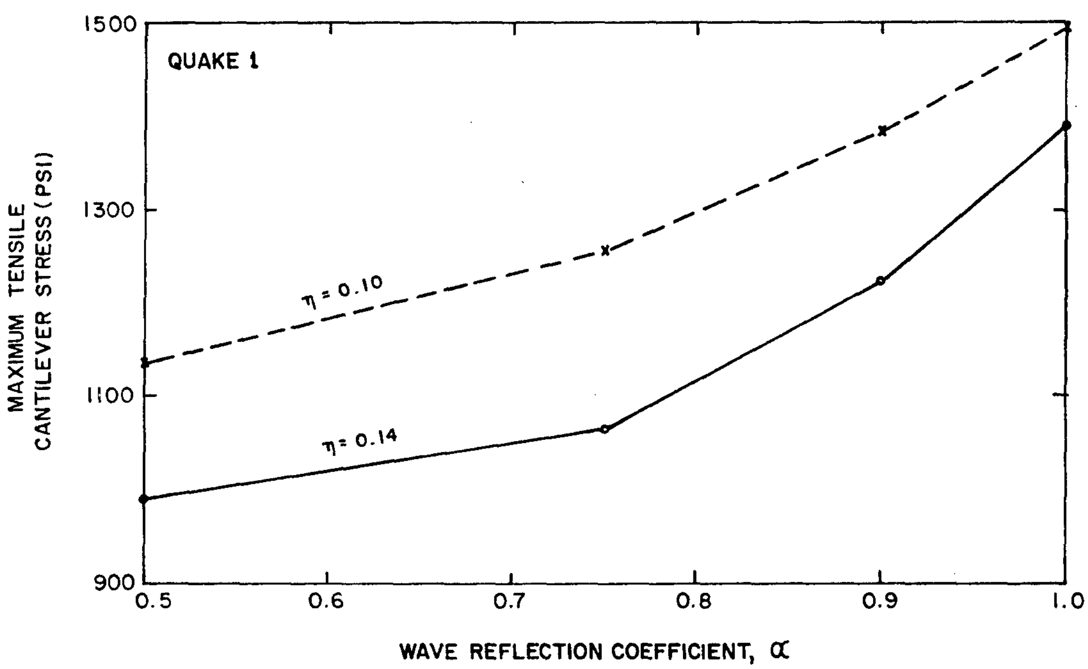
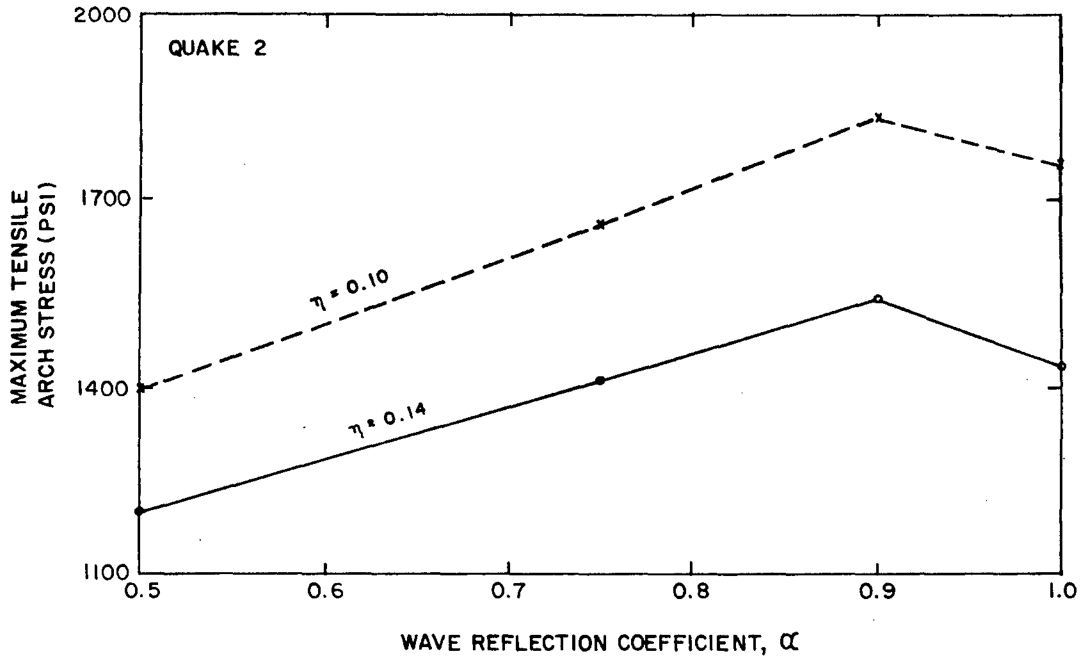


FIGURE 9-6 MAXIMUM TENSILE STRESSES (DUE TO QUAKE I EXCITATION) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) VERSUS α FOR $\eta = 0.10$ AND $\eta = 0.14$. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

STRESSES COMPUTED AT GAUSS QUADRATURE POINTS



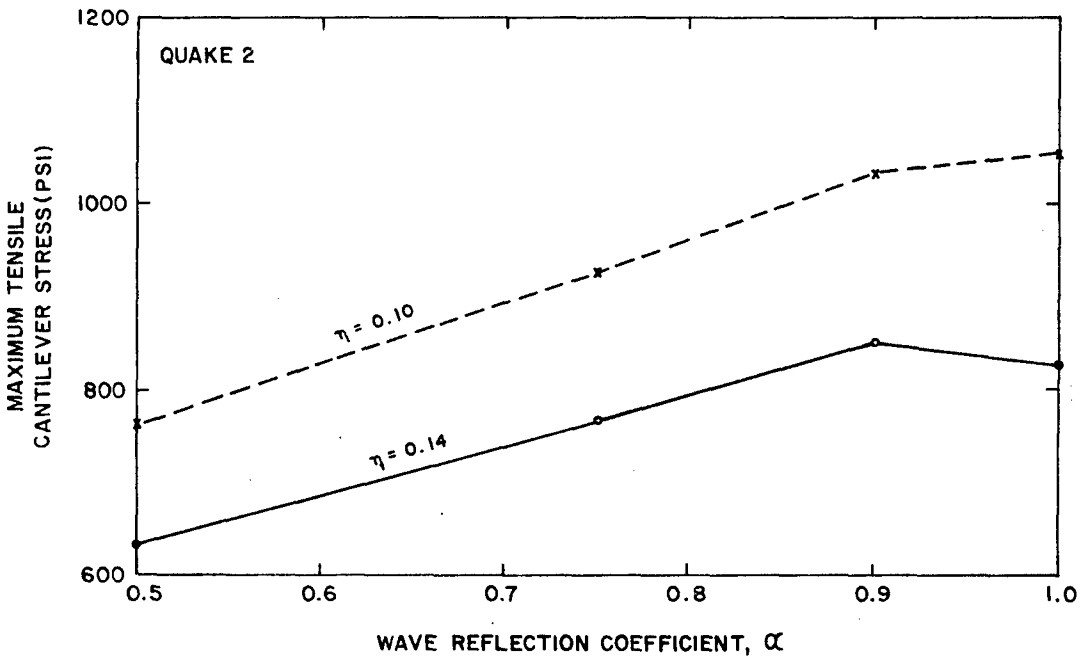
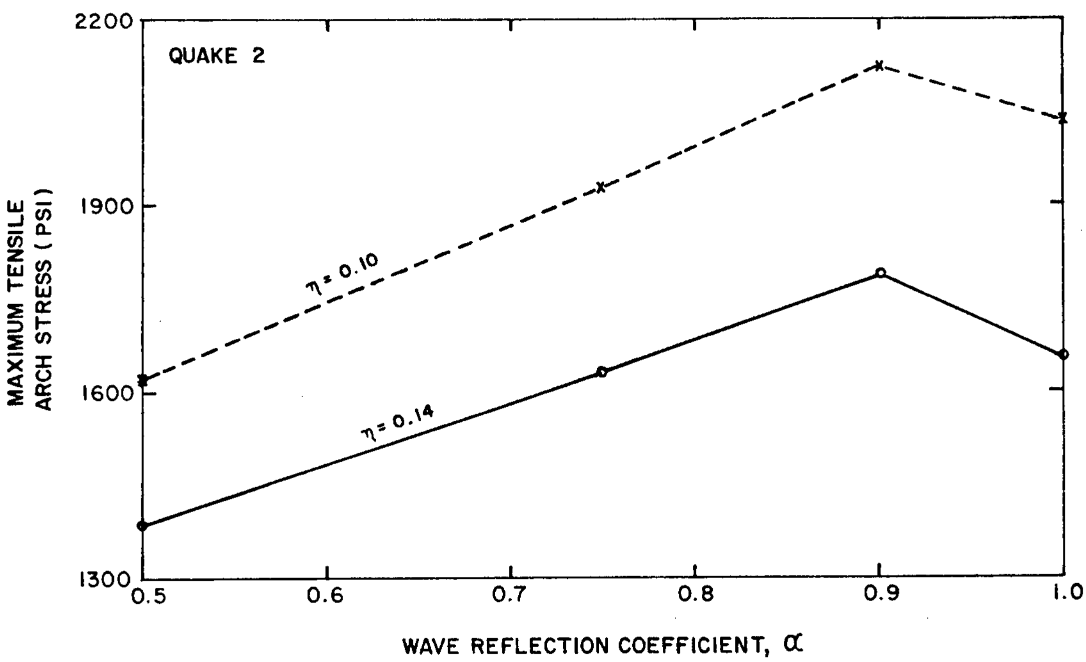


FIGURE 9-7 MAXIMUM TENSILE STRESSES (DUE TO QUAKE 2 EXCITATION) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) VERSUS α FOR $\eta = 0.10$ AND $\eta = 0.14$. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

STRESSES COMPUTED AT DAM NODAL POINTS



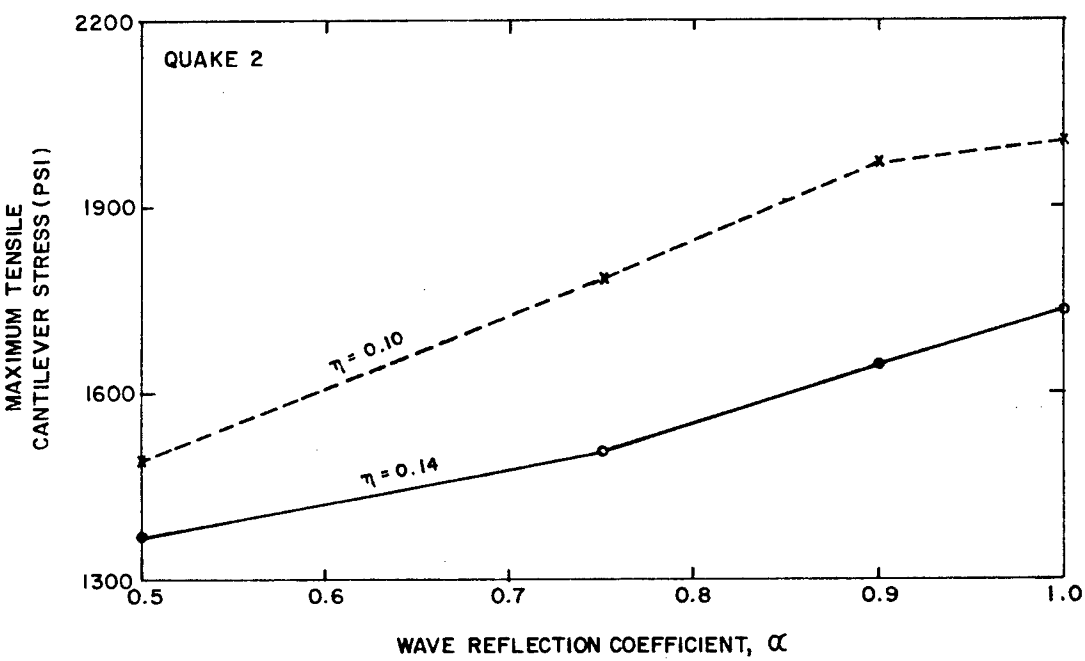


FIGURE 9-8 MAXIMUM TENSILE STRESSES (DUE TO QUAKE 2 EXCITATION) IN THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) VERSUS α FOR $\eta = 0.10$ AND $\eta = 0.14$. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

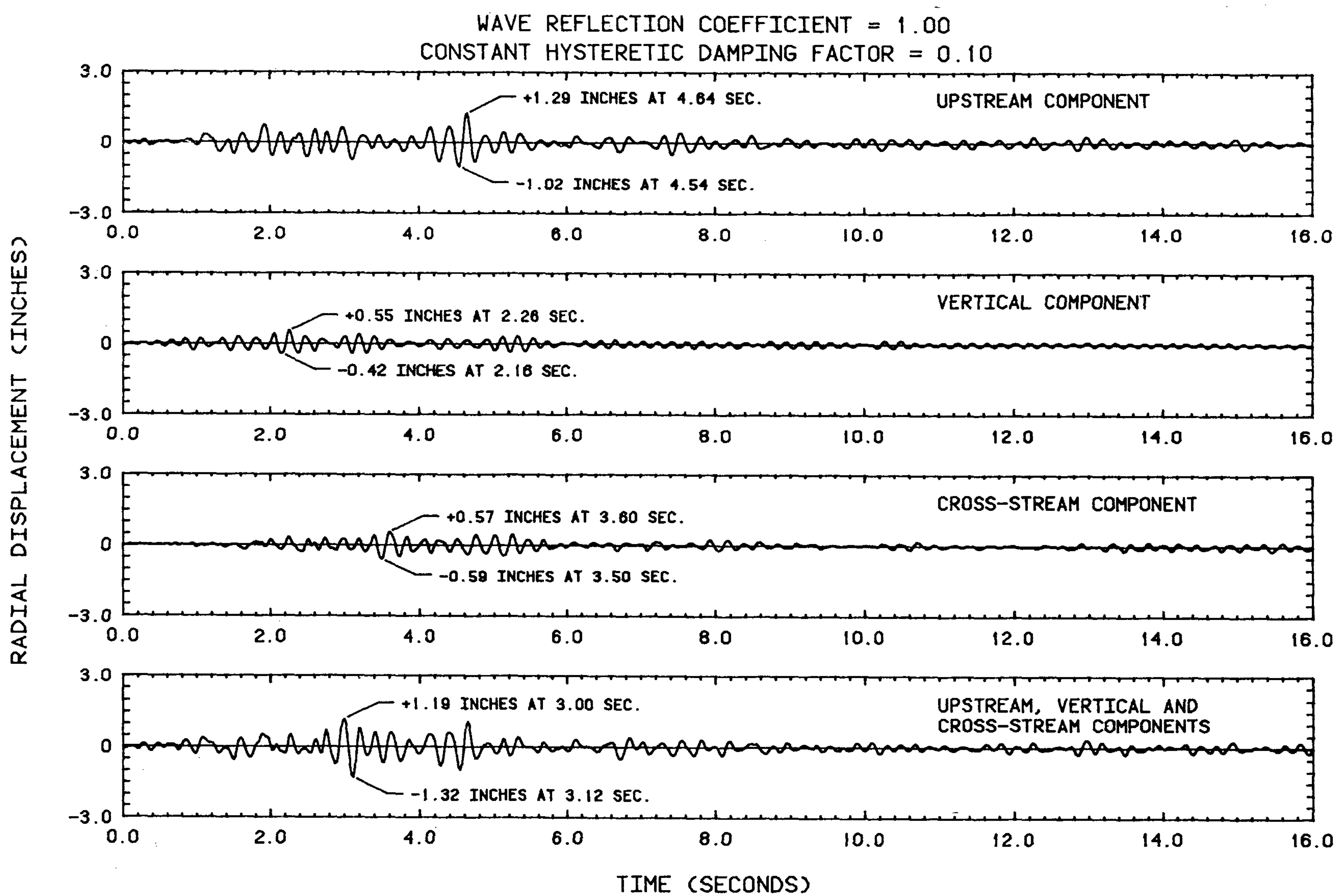


FIGURE 9-9 DYNAMIC RADIAL DISPLACEMENT RESPONSE AT DAM CREST NODAL POINT 300 DUE TO UPSTREAM, VERTICAL, AND CROSS-STREAM COMPONENTS, SEPARATELY AND SIMULTANEOUSLY, OF QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

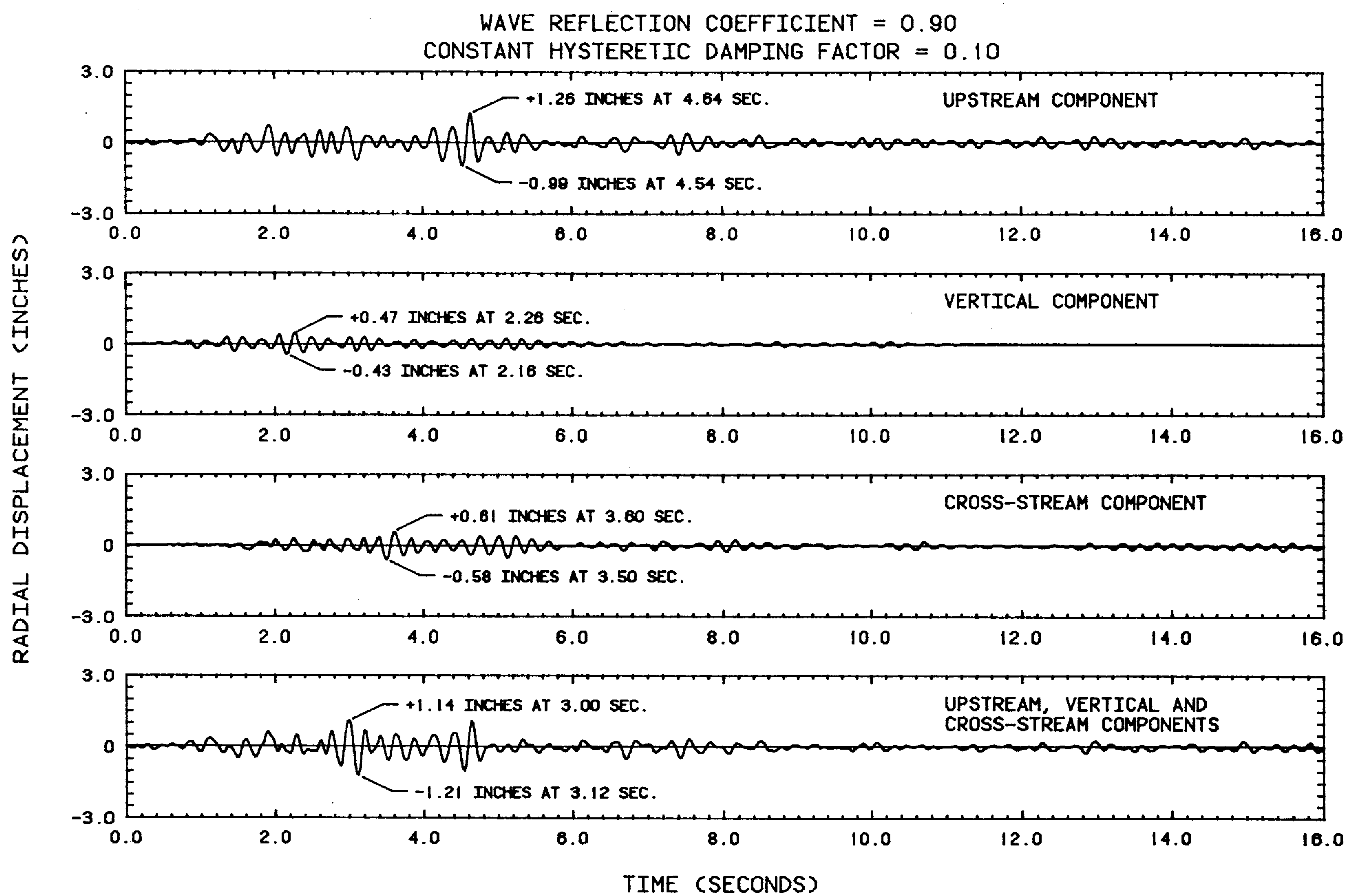


FIGURE 9-10 DYNAMIC RADIAL DISPLACEMENT RESPONSE AT DAM CREST NODAL POINT 300 DUE TO UPSTREAM, VERTICAL, AND CROSS-STREAM COMPONENTS, SEPARATELY AND SIMULTANEOUSLY, OF QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

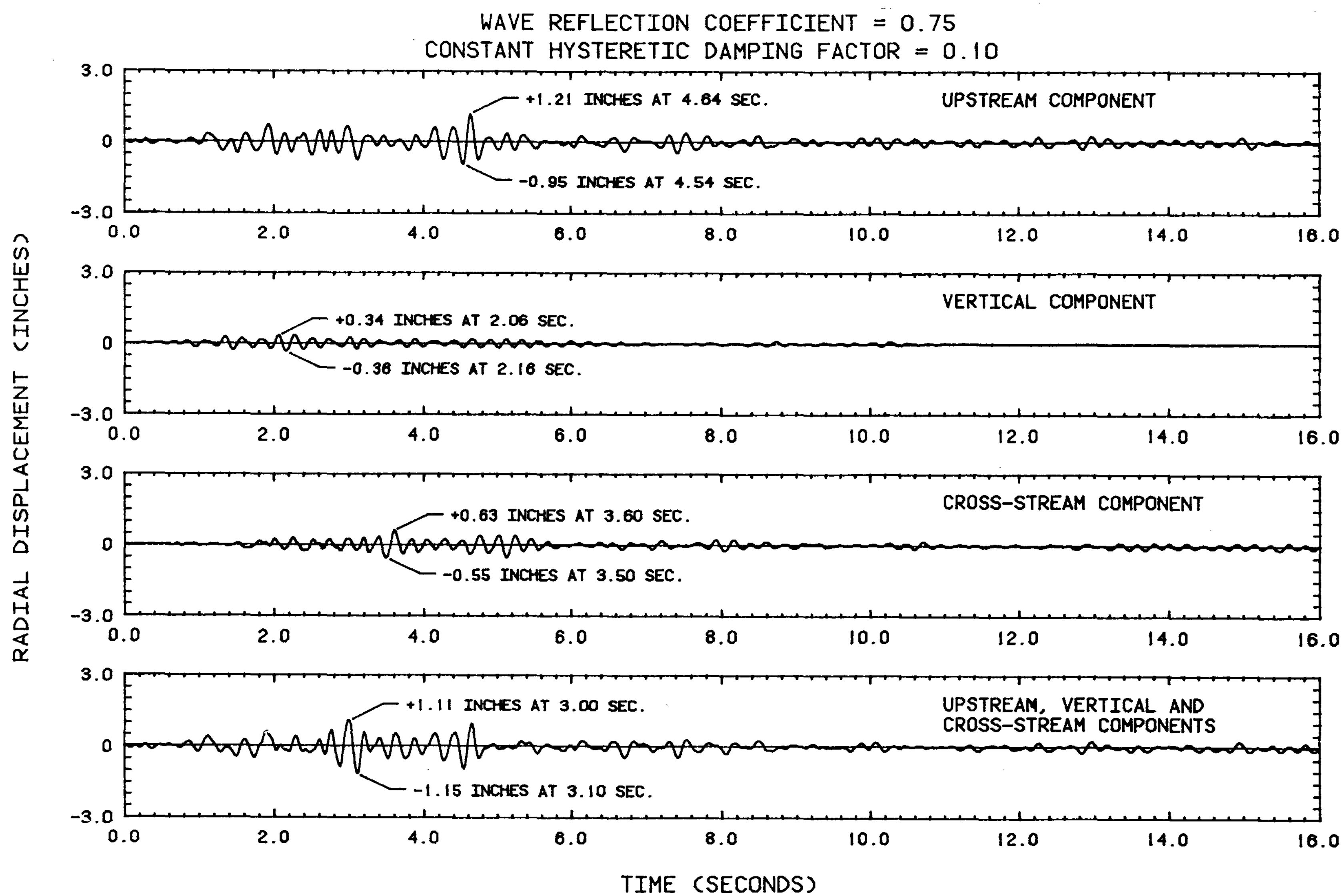
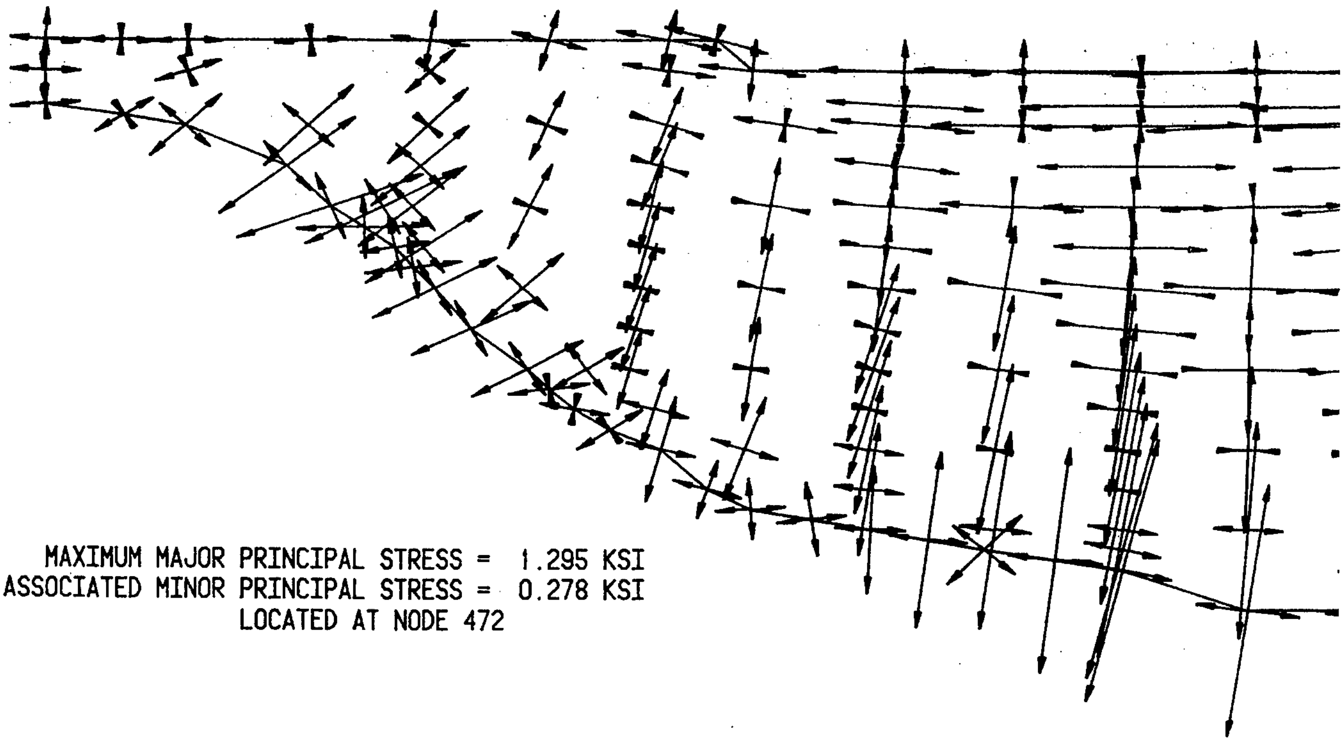
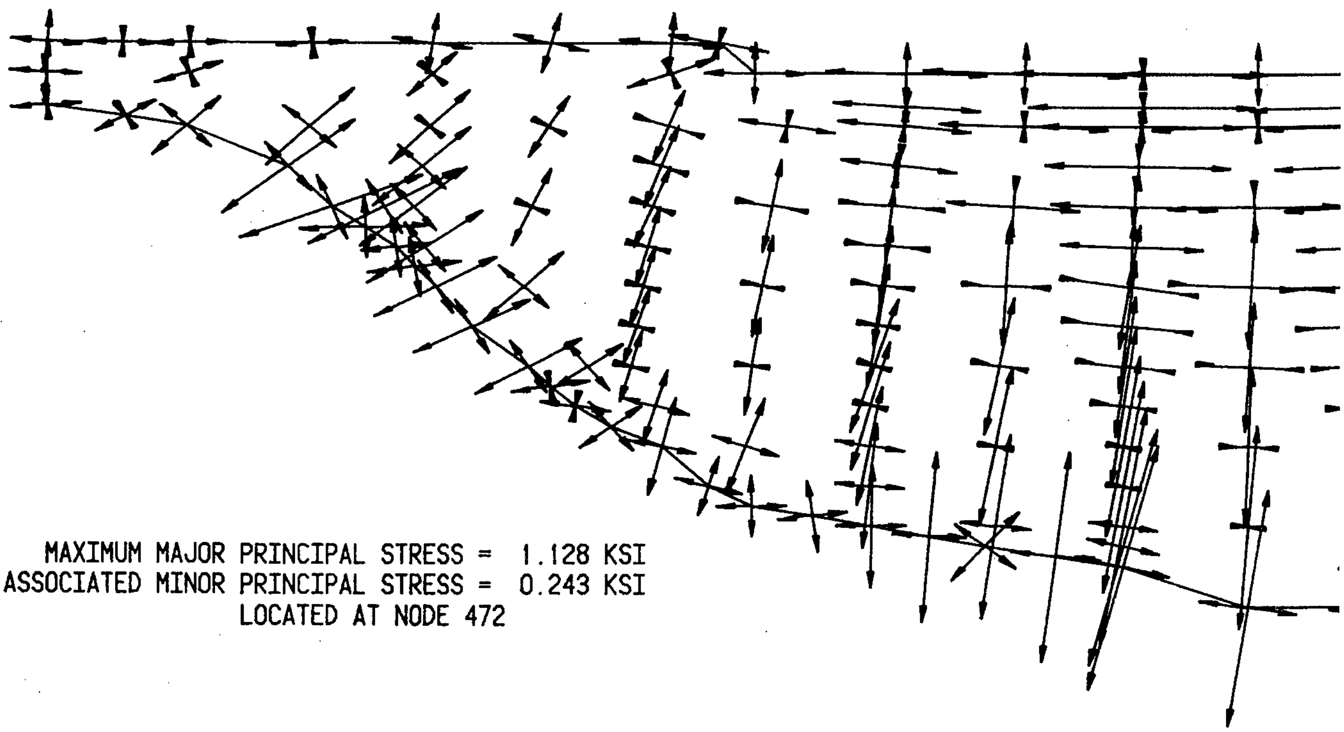


FIGURE 9-11 DYNAMIC RADIAL DISPLACEMENT RESPONSE AT DAM CREST NODAL POINT 300 DUE TO UPSTREAM, VERTICAL, AND CROSS-STREAM COMPONENTS, SEPARATELY AND SIMULTANEOUSLY, OF QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).



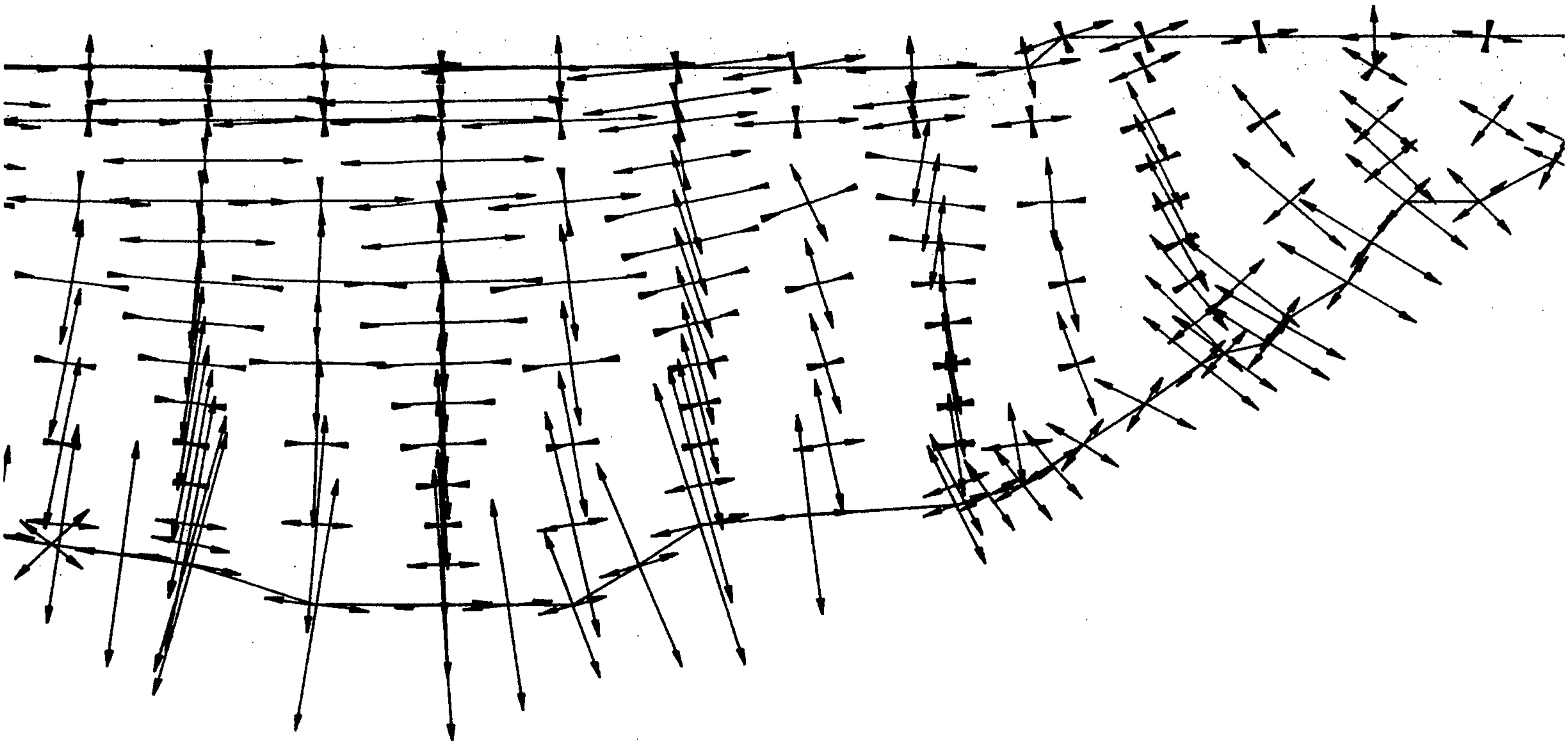
MAXIMUM MAJOR PRINCIPAL STRESS = 1.295 KSI
ASSOCIATED MINOR PRINCIPAL STRESS = 0.278 KSI
LOCATED AT NODE 472

FIGURE 9-12 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AN
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RE
STATIC STRESSES DUE TO DEAD WEIGHT OF T
WAVE REFLECTIO

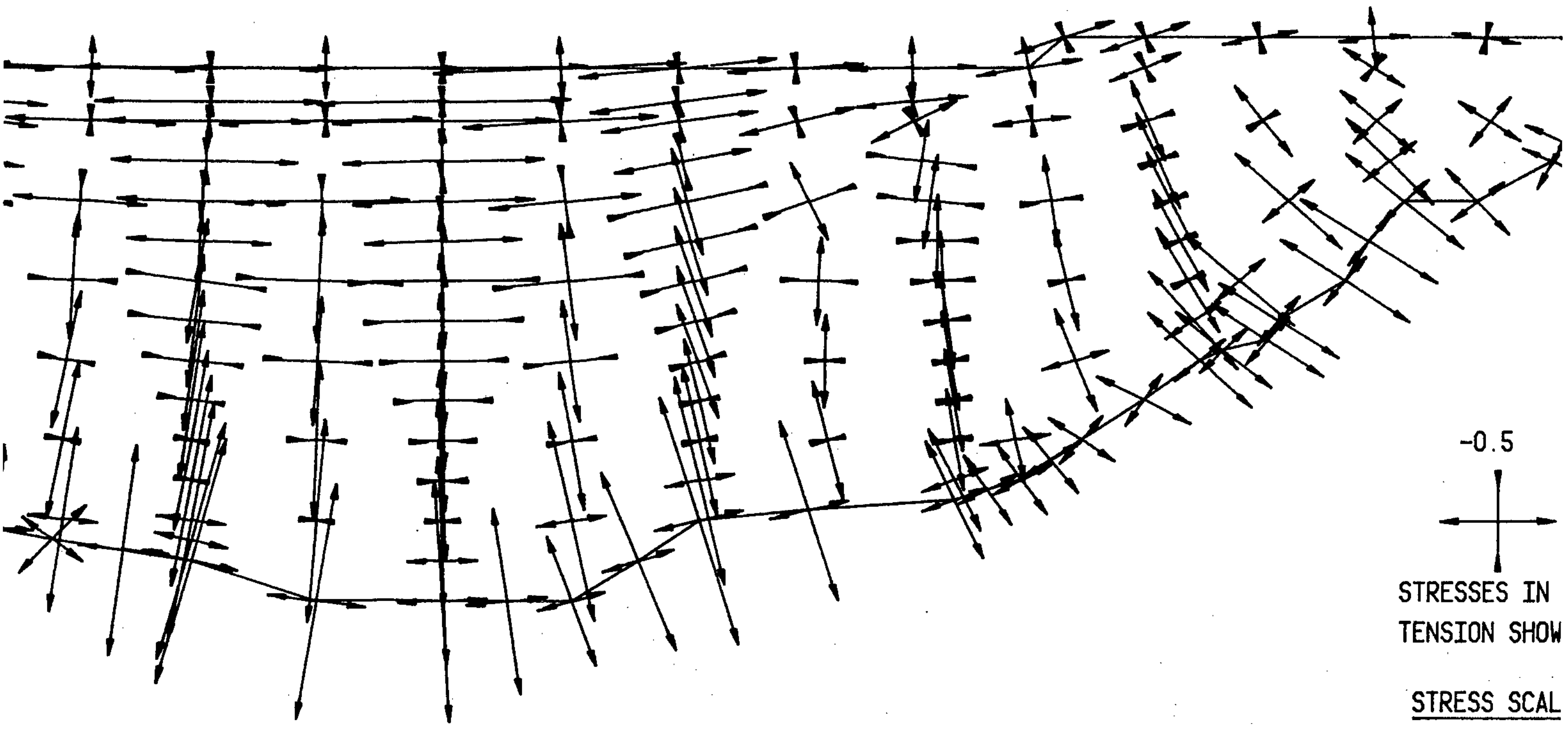


MAXIMUM MAJOR PRINCIPAL STRESS = 1.128 KSI
ASSOCIATED MINOR PRINCIPAL STRESS = 0.243 KSI
LOCATED AT NODE 472

FIGURE 9-13 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AN
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RE
STATIC STRESSES DUE TO DEAD WEIGHT OF T
WAVE REFLECTIO

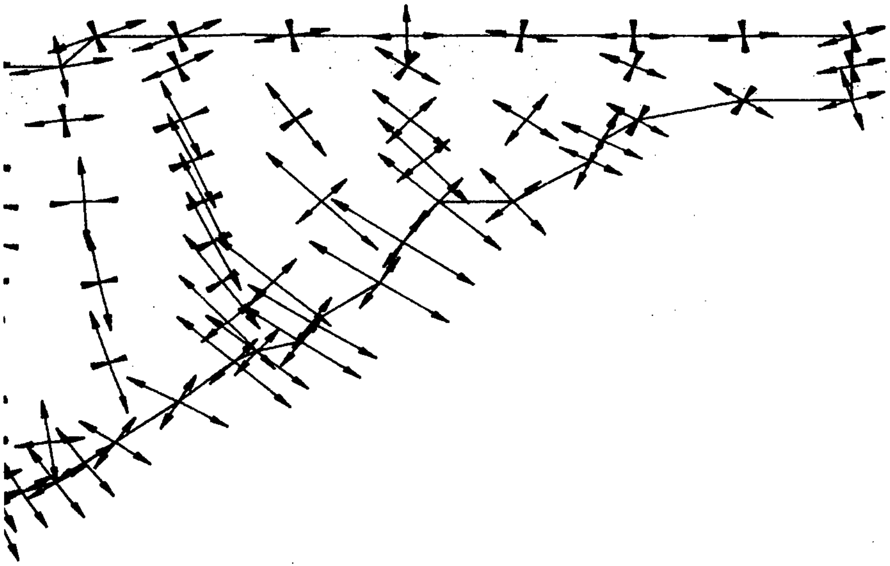


F MAJOR PRINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON UPSTREAM FACE OF THE
FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1.
ESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.90

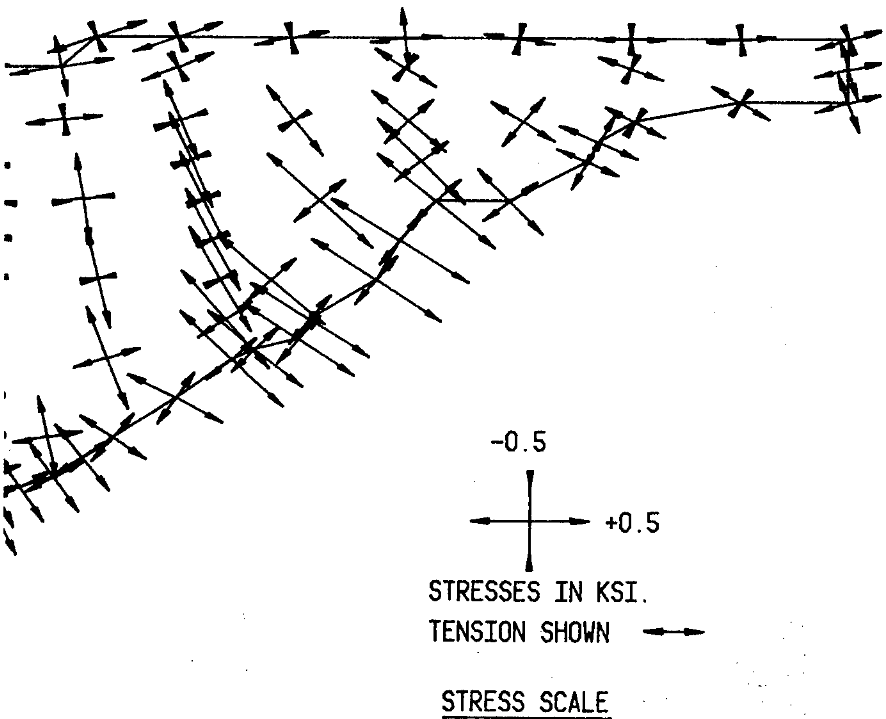


F MAJOR PRINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON UPSTREAM FACE OF THE
FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1.
ESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.75

2



S ON UPSTREAM FACE OF THE
TO QUAKE 1.
UDED.



S ON UPSTREAM FACE OF THE
TO QUAKE 1.
JDED.

3

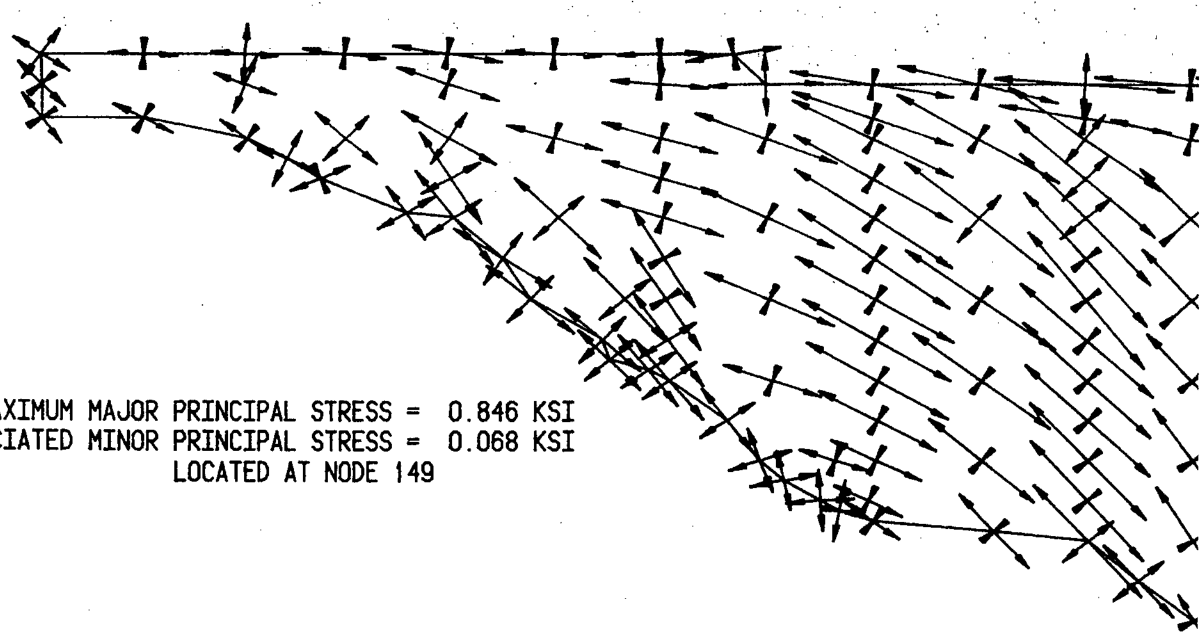


FIGURE 9-14 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AND
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RES
STATIC STRESSES DUE TO DEAD WEIGHT OF TH
WAVE REFLECTION

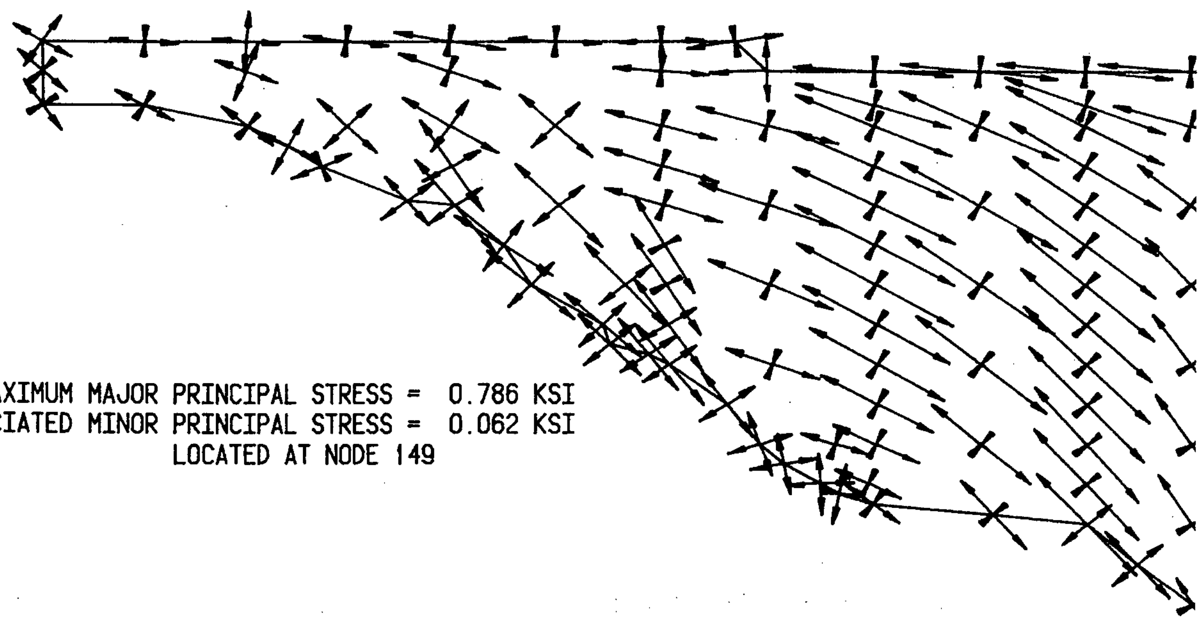
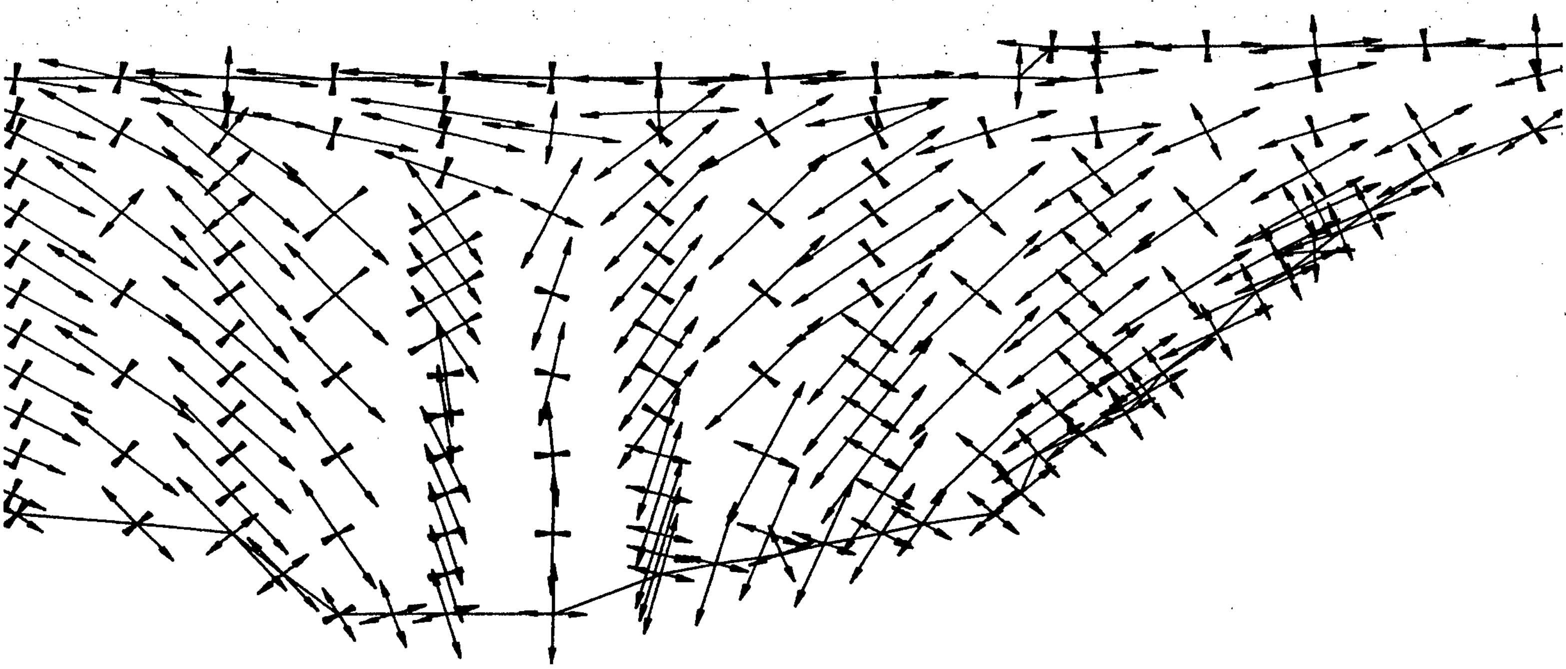
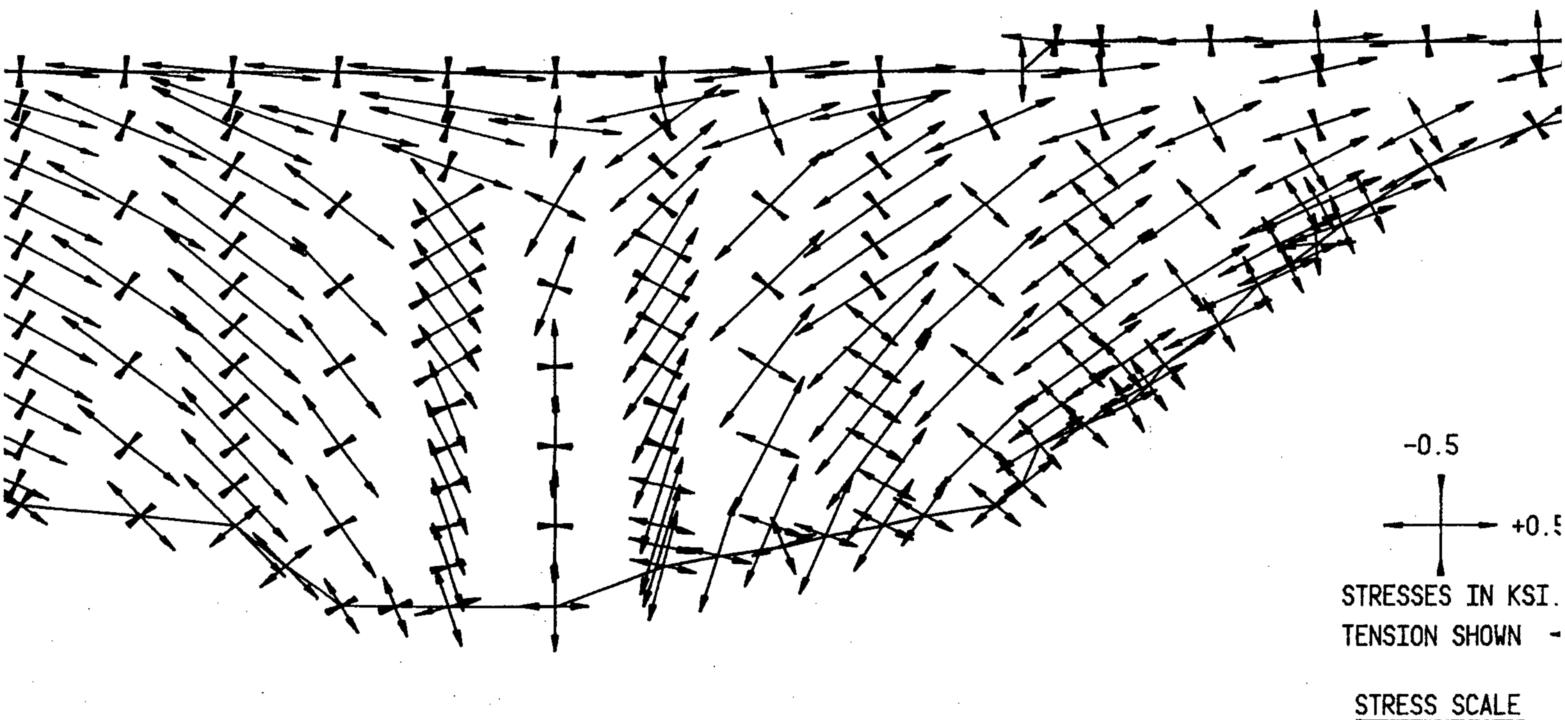


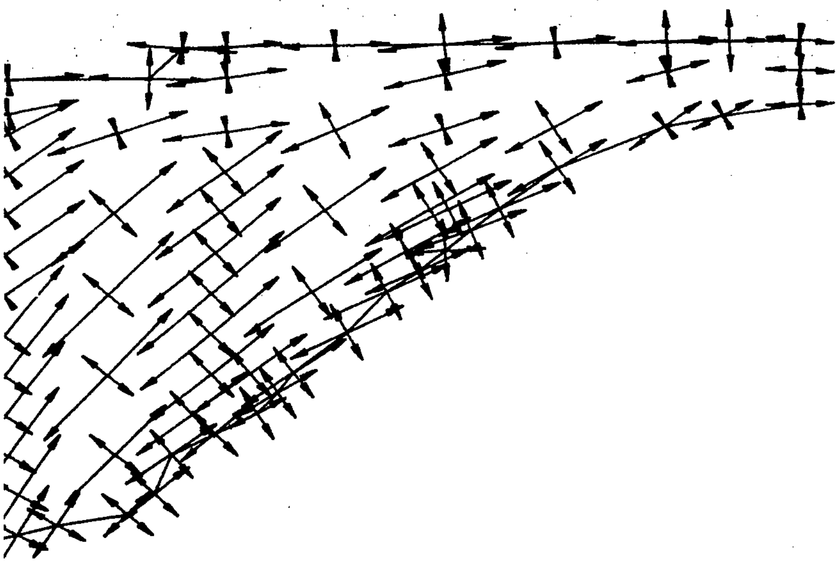
FIGURE 9-15 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AND
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RES
STATIC STRESSES DUE TO DEAD WEIGHT OF TH
WAVE REFLECTION



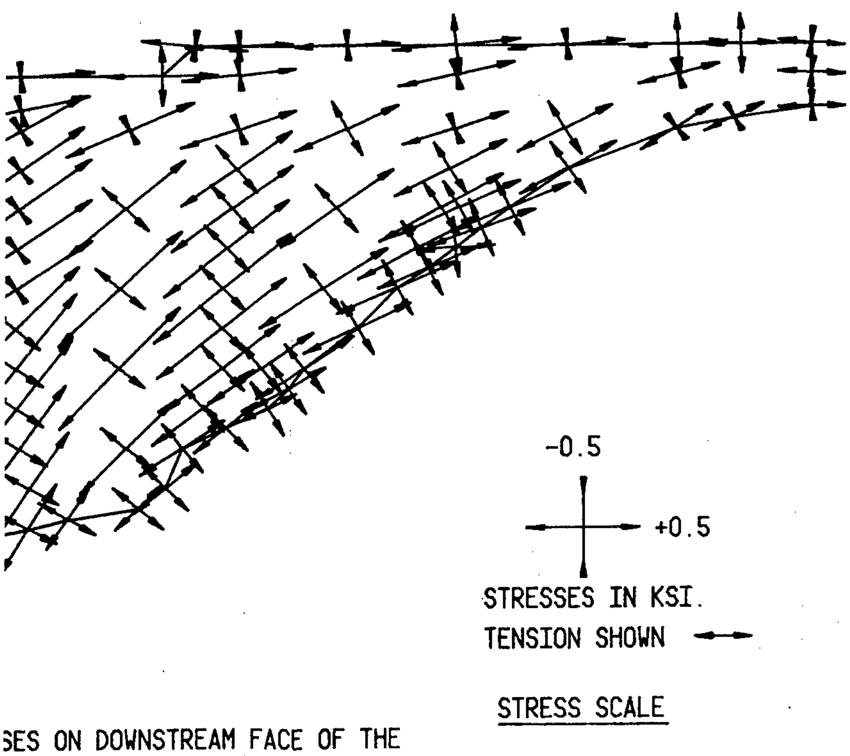
R PRINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE
DATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1.
DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.90



IR PRINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE
IDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1.
; DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.75

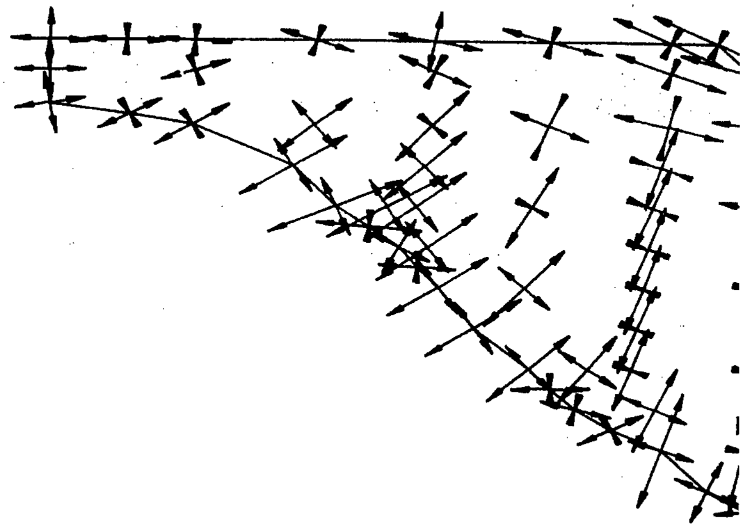


ES ON DOWNSTREAM FACE OF THE
E TO QUAKE 1.
CLUDED.



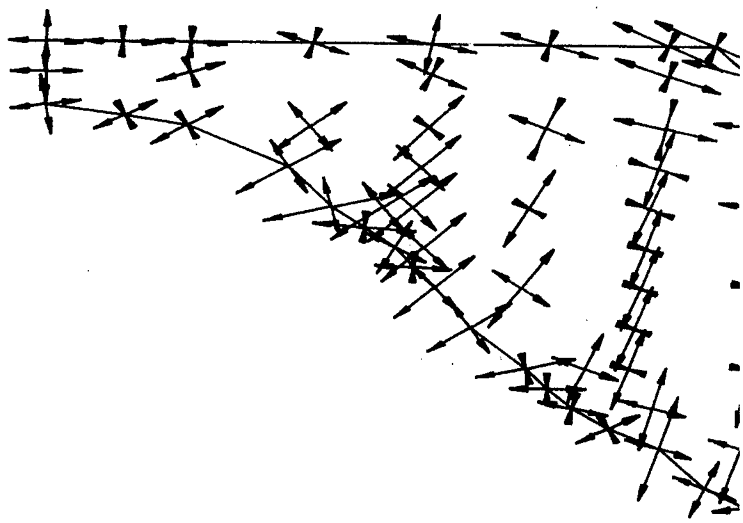
3ES ON DOWNSTREAM FACE OF THE
JE TO QUAKE 1.
NCLUDED.

3



MAXIMUM MAJOR PRINCIPAL STRESS = 1.791 KSI
ASSOCIATED MINOR PRINCIPAL STRESS = -0.006 KSI
LOCATED AT NODE 303

FIGURE 9-16 ENV
D



MAXIMUM MAJOR PRINCIPAL STRESS = 1.638 KSI
ASSOCIATED MINOR PRINCIPAL STRESS = -0.006 KSI
LOCATED AT NODE 303

FIGURE 9-17 ENV
D

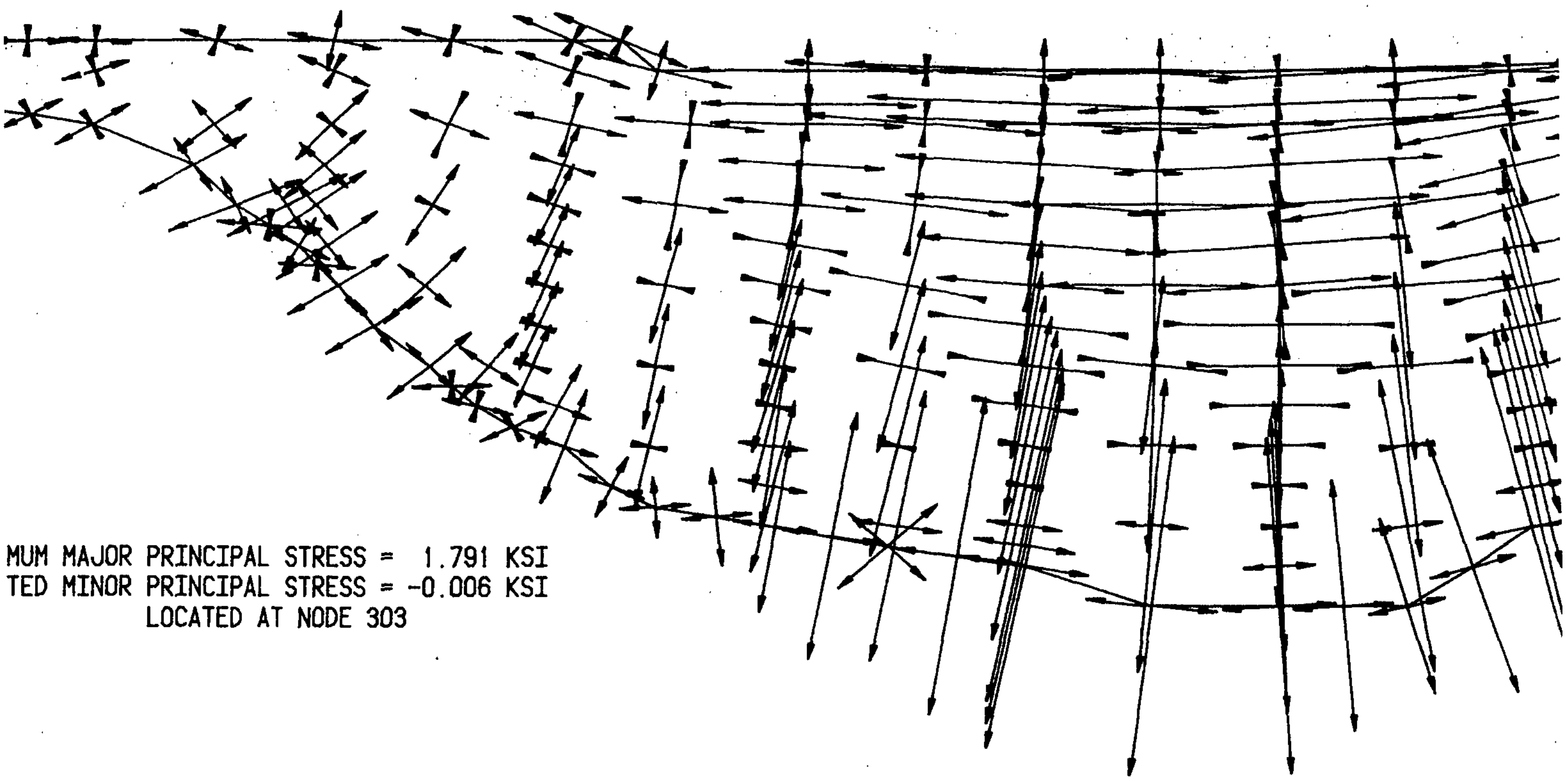


FIGURE 9-16 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AND THE ASSOCIATED MINOR I
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION
STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC I
WAVE REFLECTION COEFFICIENT = 0.90

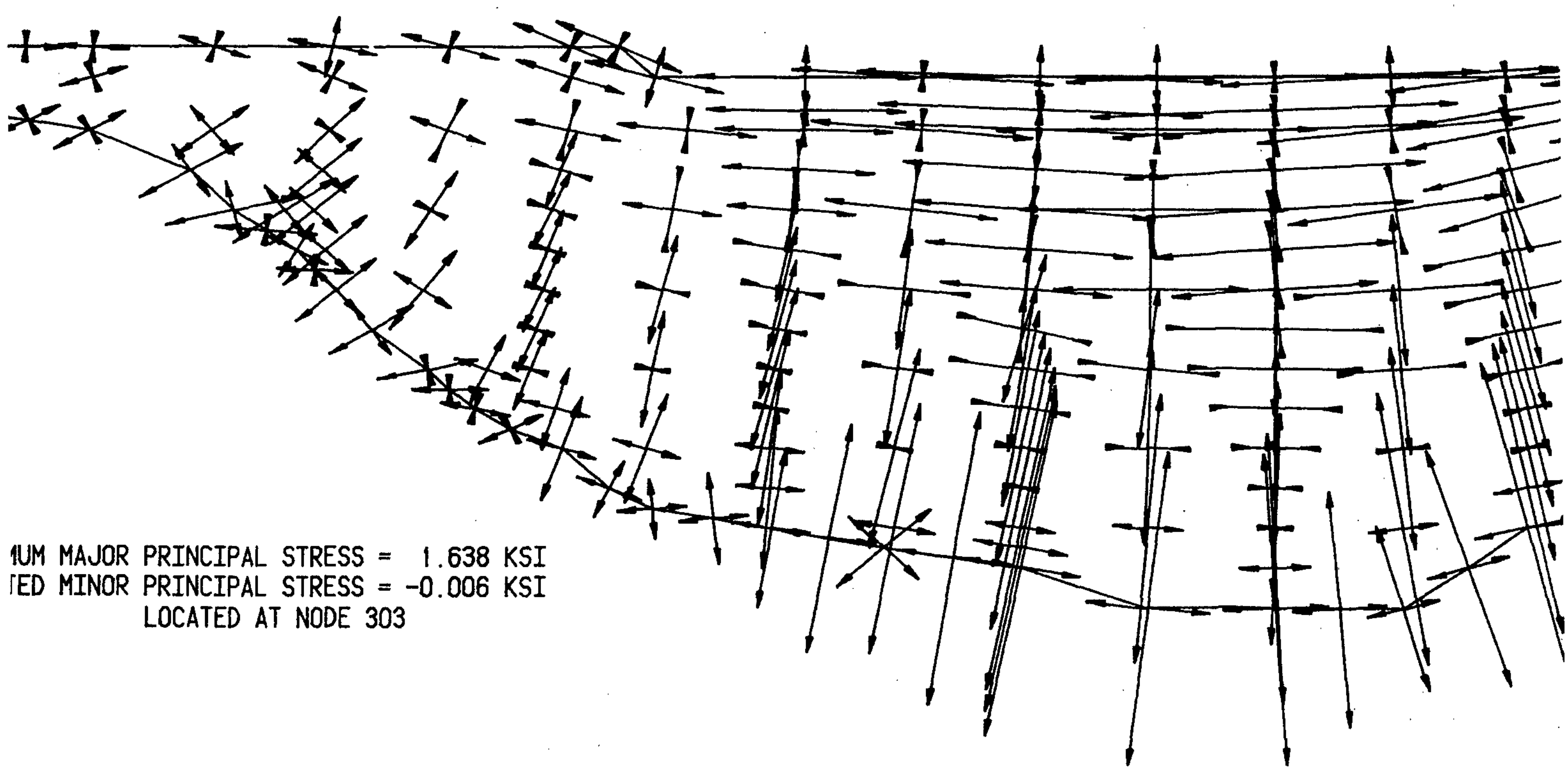
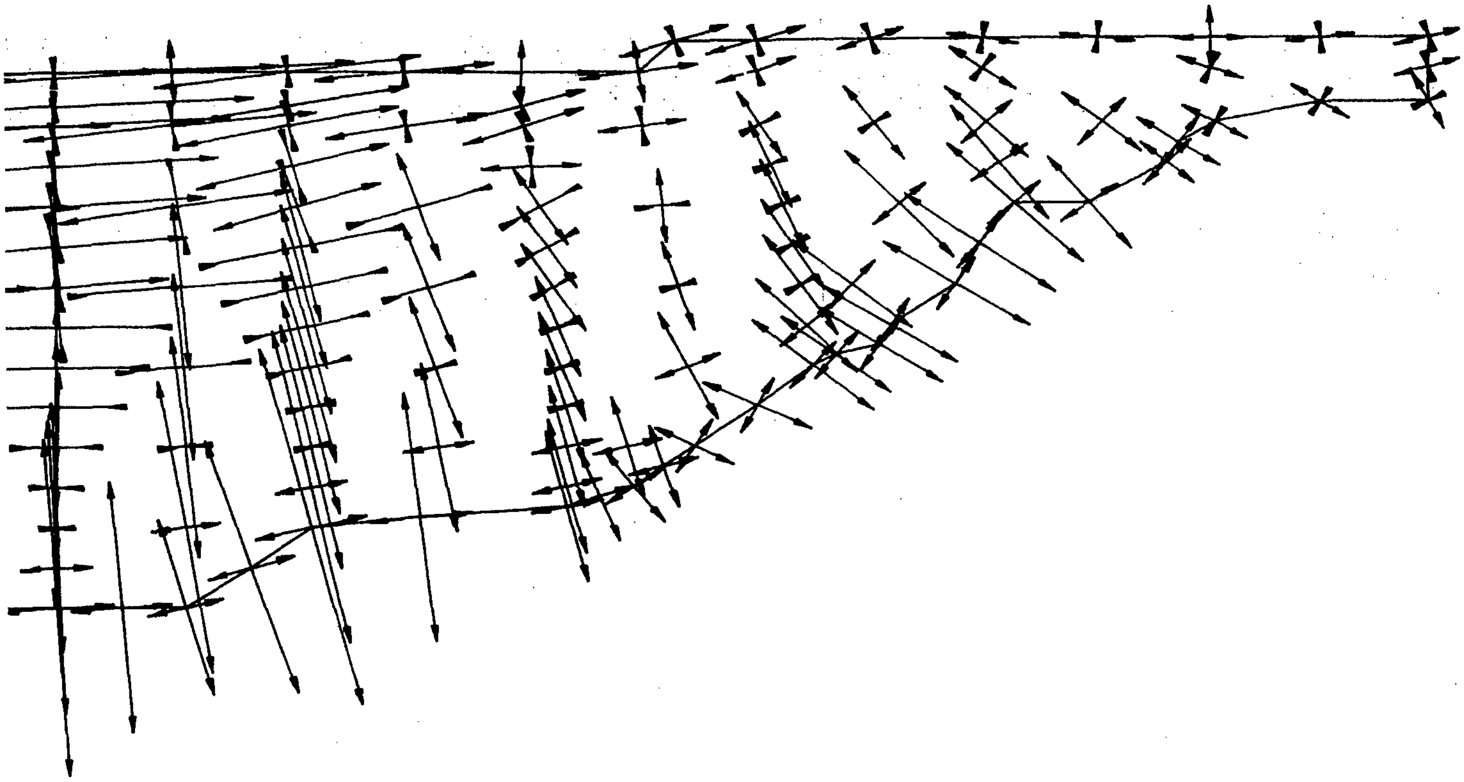
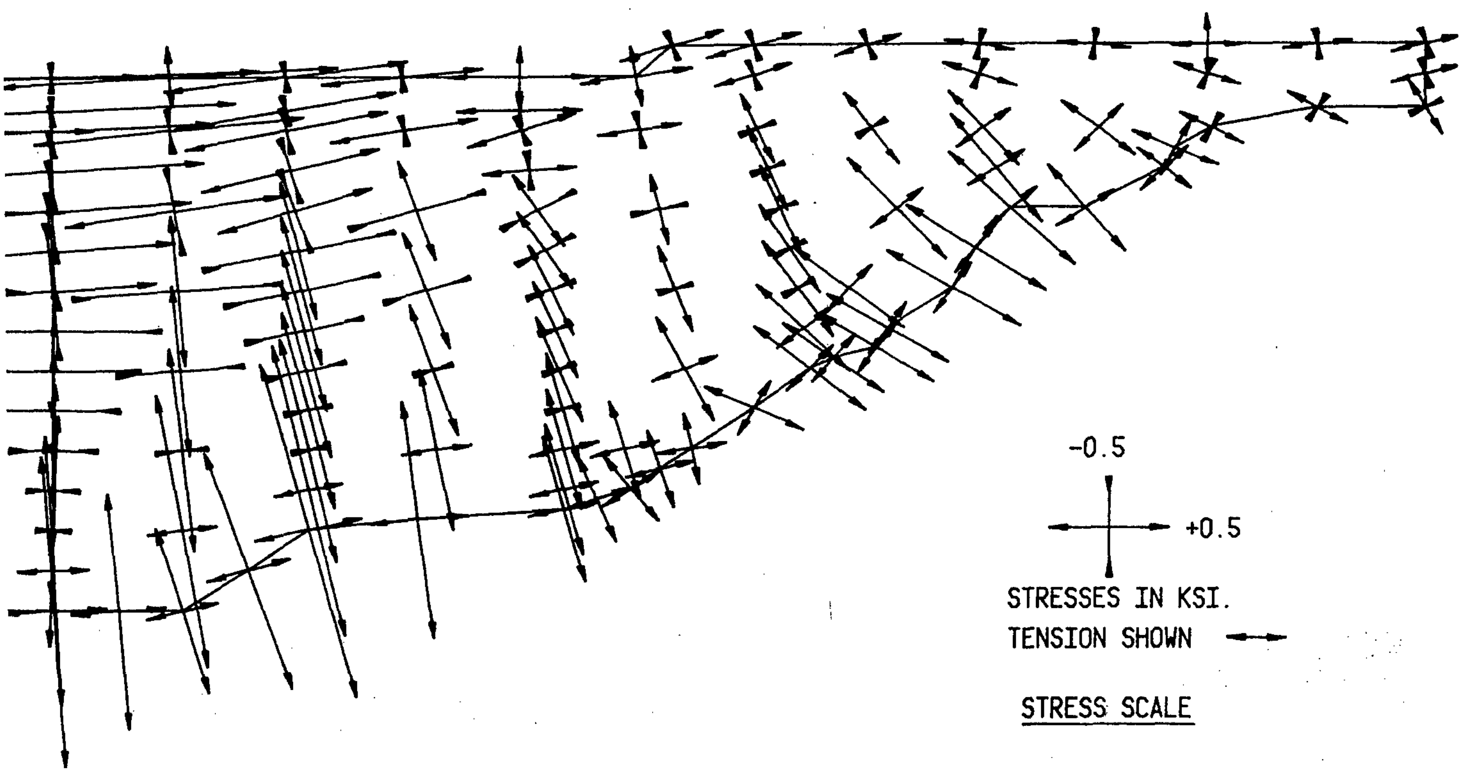


FIGURE 9-17 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AND THE ASSOCIATED MINOR F
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION.
STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC F
WAVE REFLECTION COEFFICIENT = 0.75



ND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON UPSTREAM FACE OF THE
ESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2.
THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
ON COEFFICIENT = 0.90



ND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON UPSTREAM FACE OF THE
ESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2.
THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
ON COEFFICIENT = 0.75

3

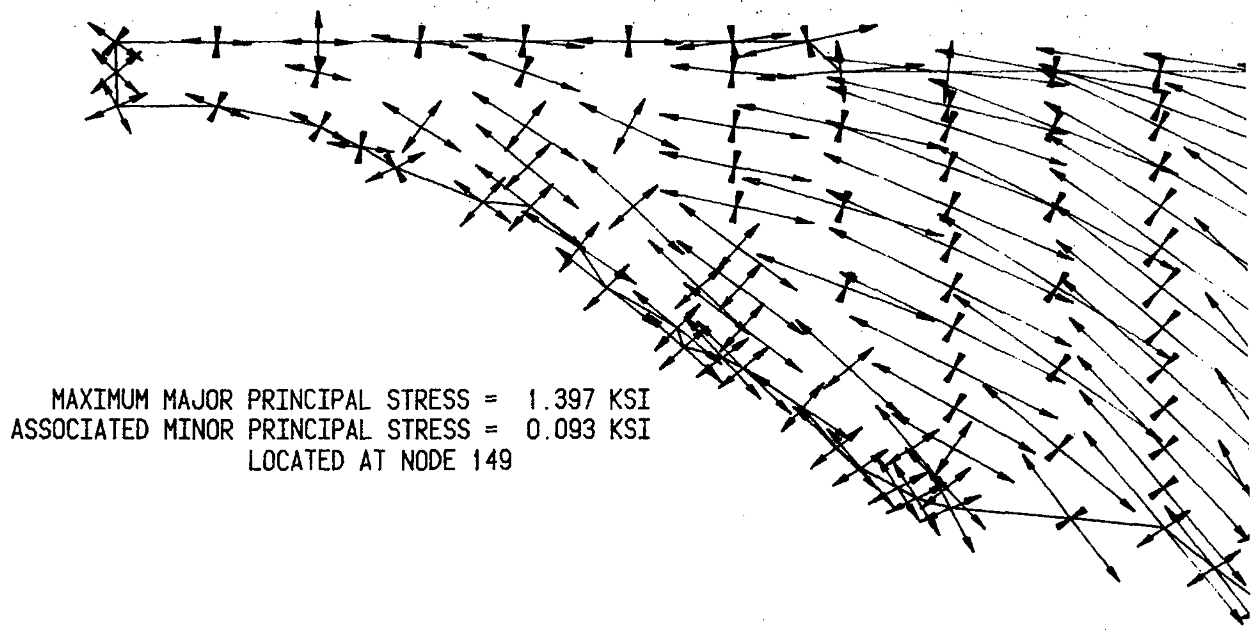


FIGURE 9-18 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AN
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL R
STATIC STRESSES DUE TO DEAD WEIGHT OF
WAVE REFLECTI

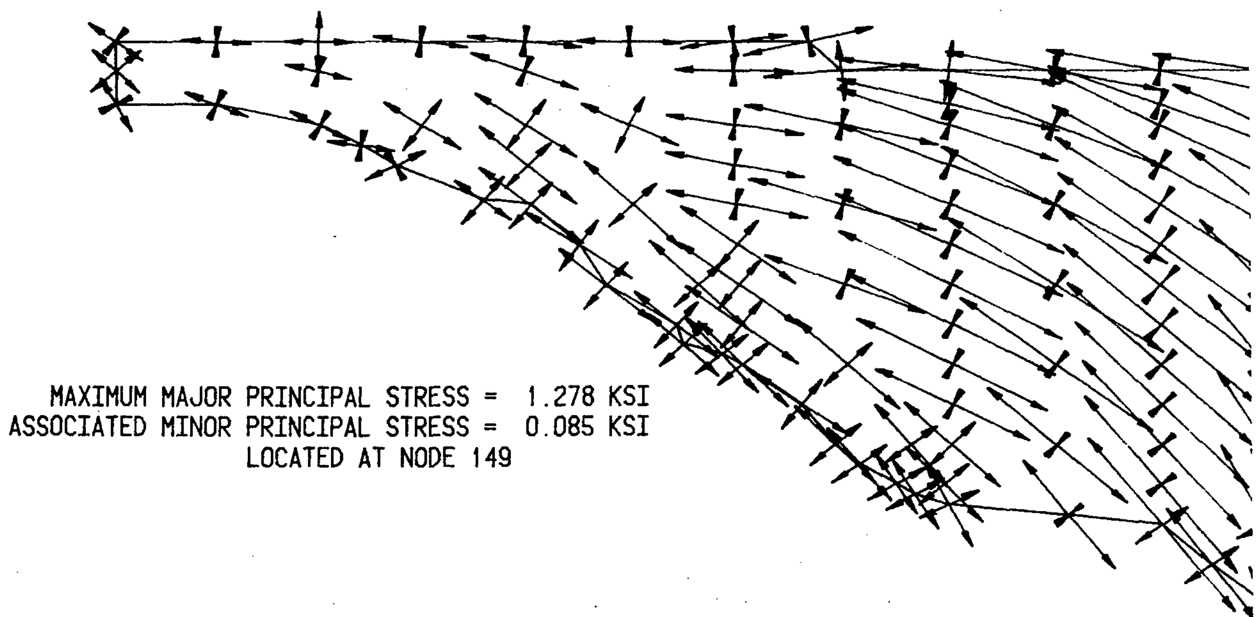
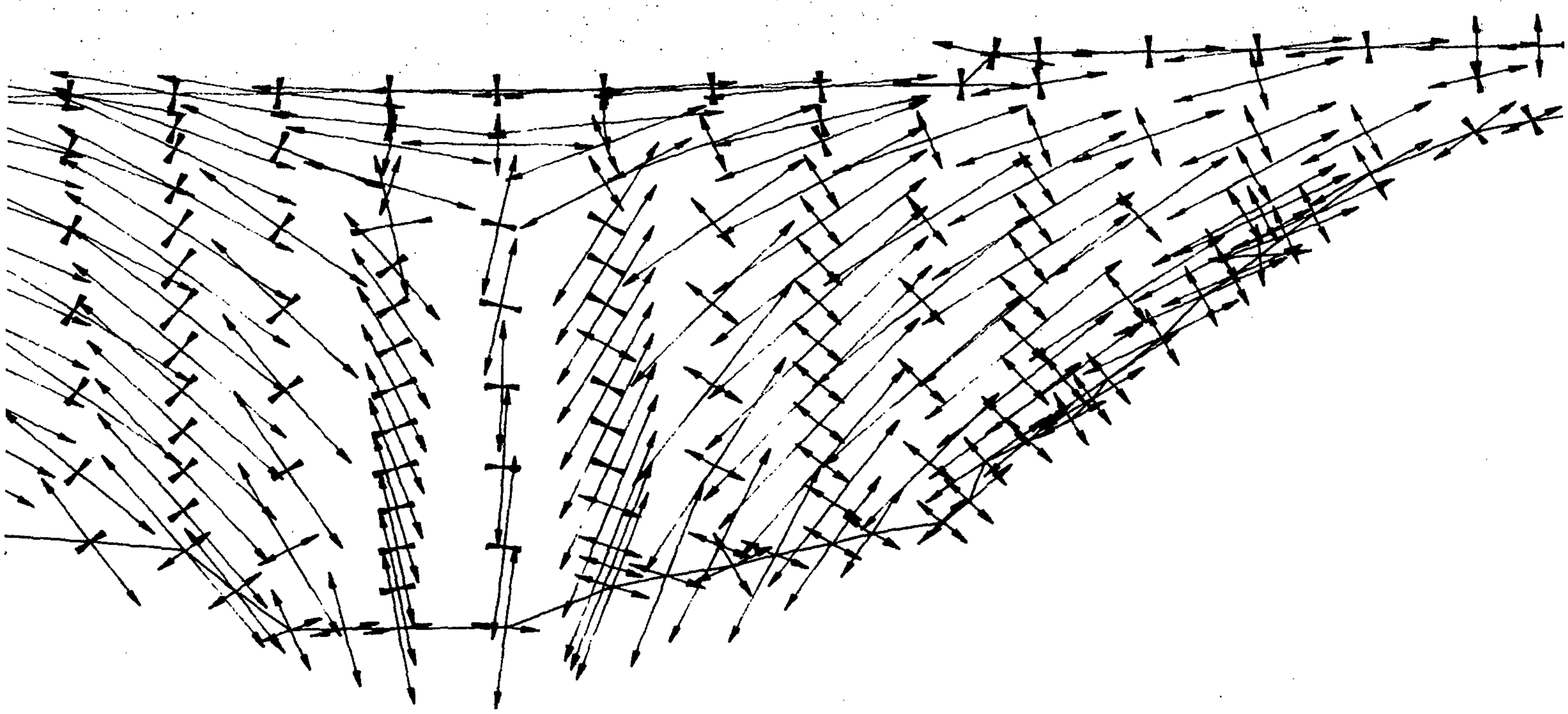
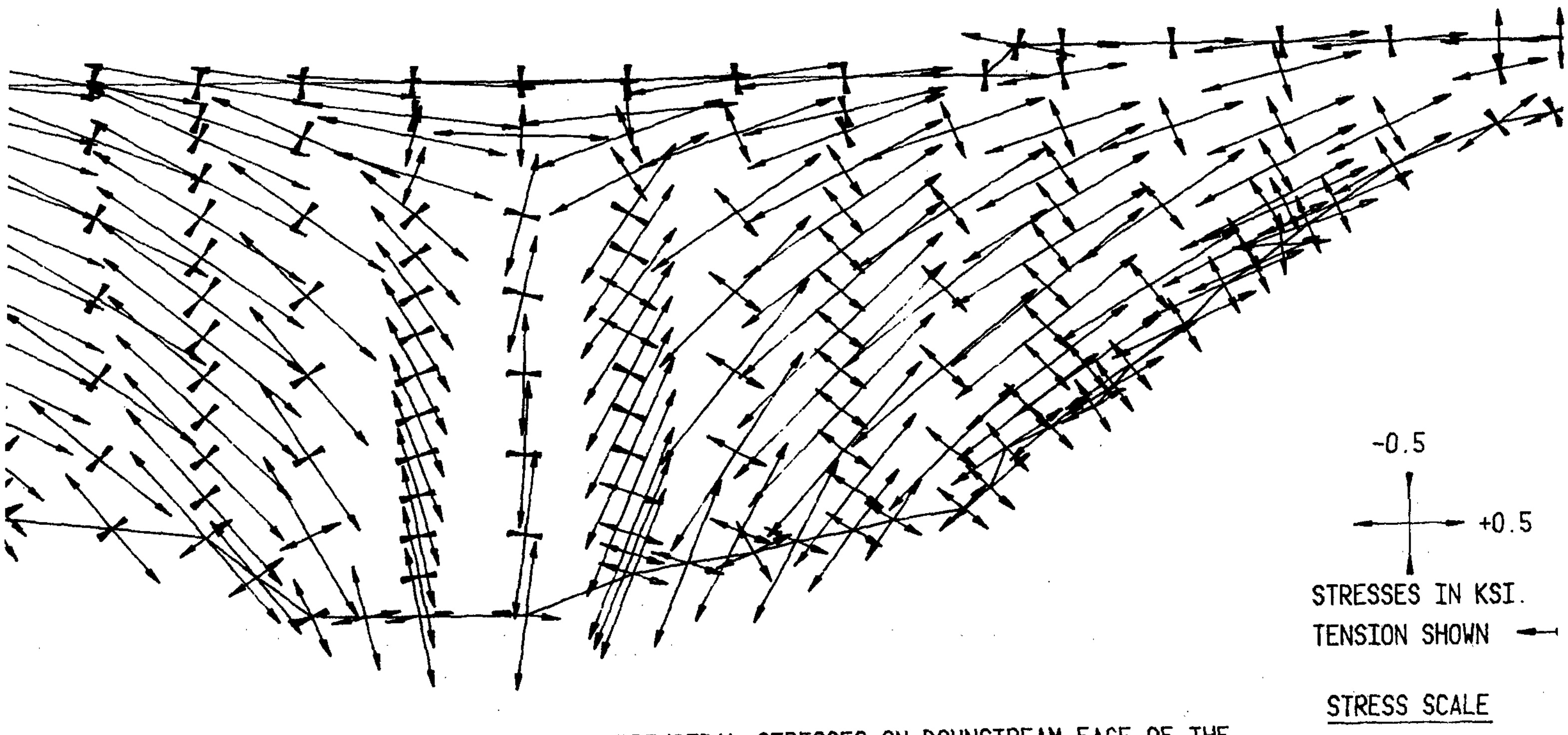


FIGURE 9-19 ENVELOPE VALUES OF MAJOR PRINCIPAL STRESSES AN[
DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL R[
STATIC STRESSES DUE TO DEAD WEIGHT OF
WAVE REFLECTI(

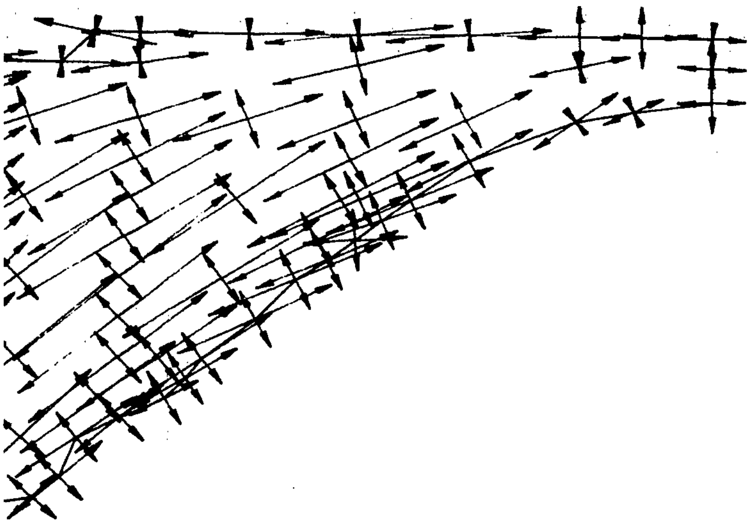


RINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE
ION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2.
E TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.90

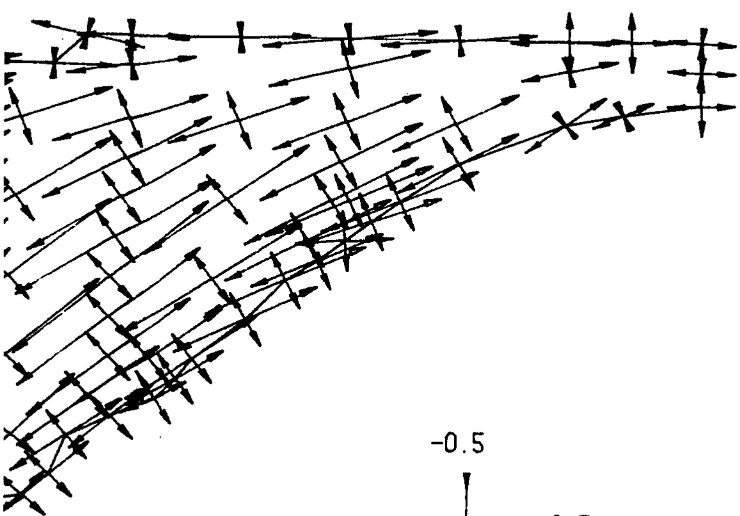


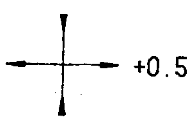
PRINCIPAL STRESSES AND THE ASSOCIATED MINOR PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE
ATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2.
DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.
WAVE REFLECTION COEFFICIENT = 0.75

2



ƆWNSTREAM FACE OF THE
AKE 2.





STRESSES IN KSI.
TENSION SHOWN

STRESS SCALE

WNSTREAM FACE OF THE
KE 2.

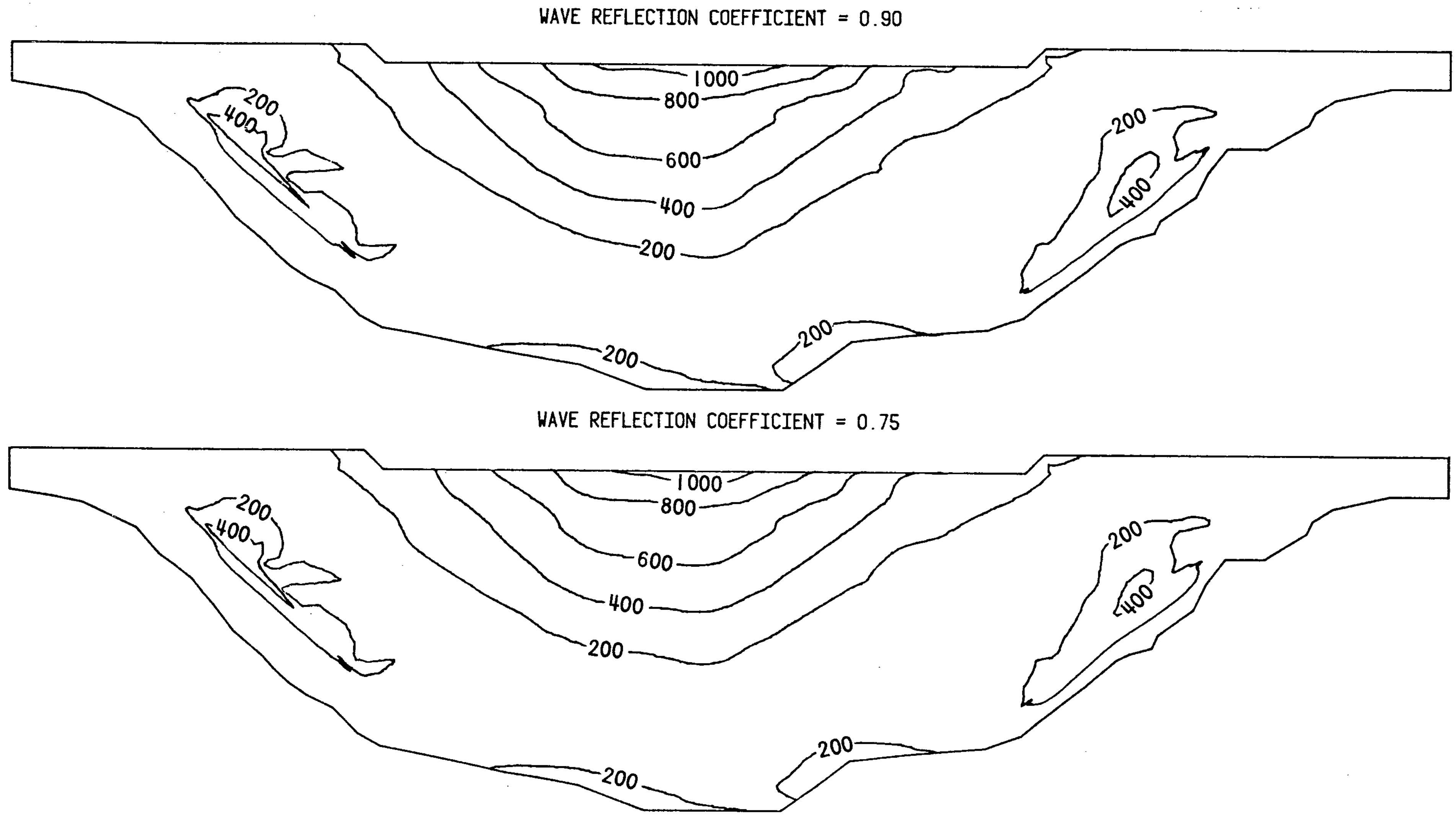
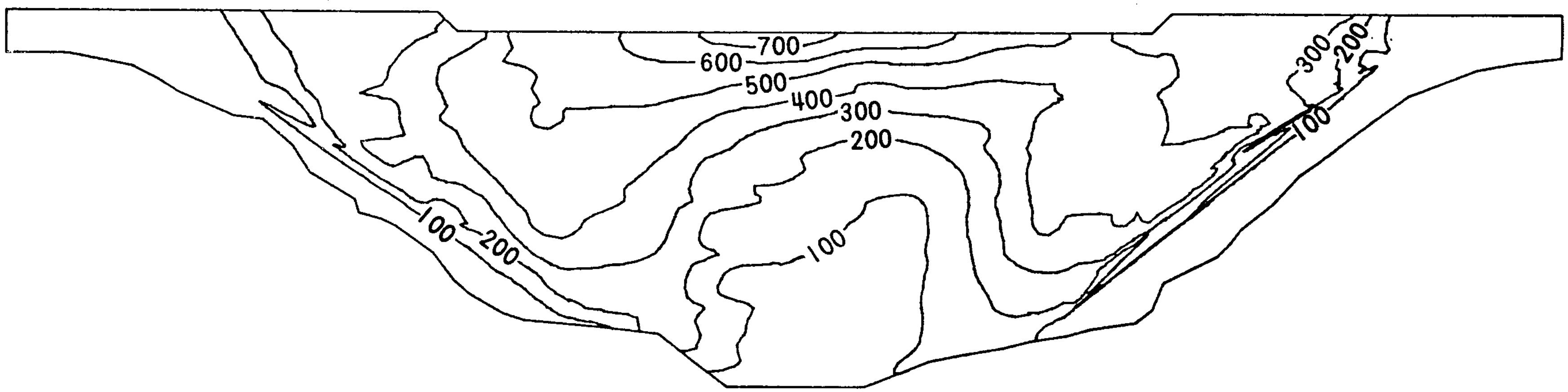


FIGURE 9-20 ENVELOPE VALUES OF MAXIMUM (TENSILE) ARCH STRESSES ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

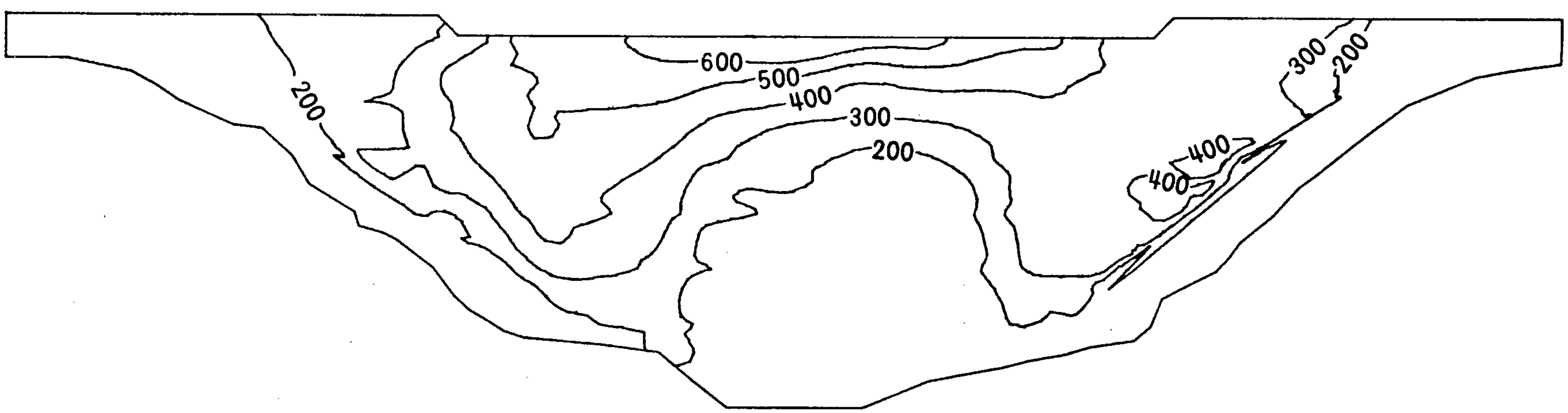


FIGURE 9-21 ENVELOPE VALUES OF MAXIMUM (TENSILE) ARCH STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

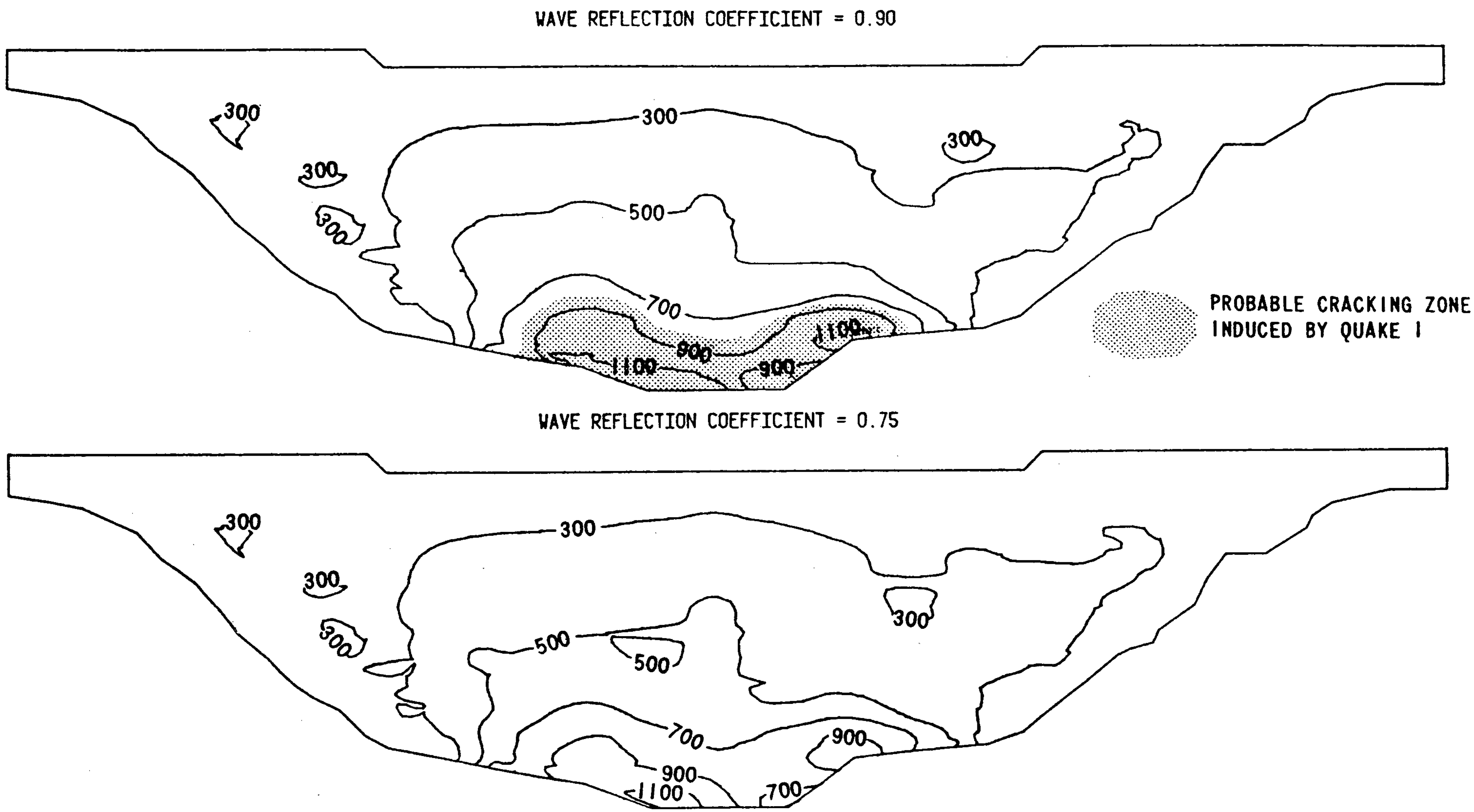
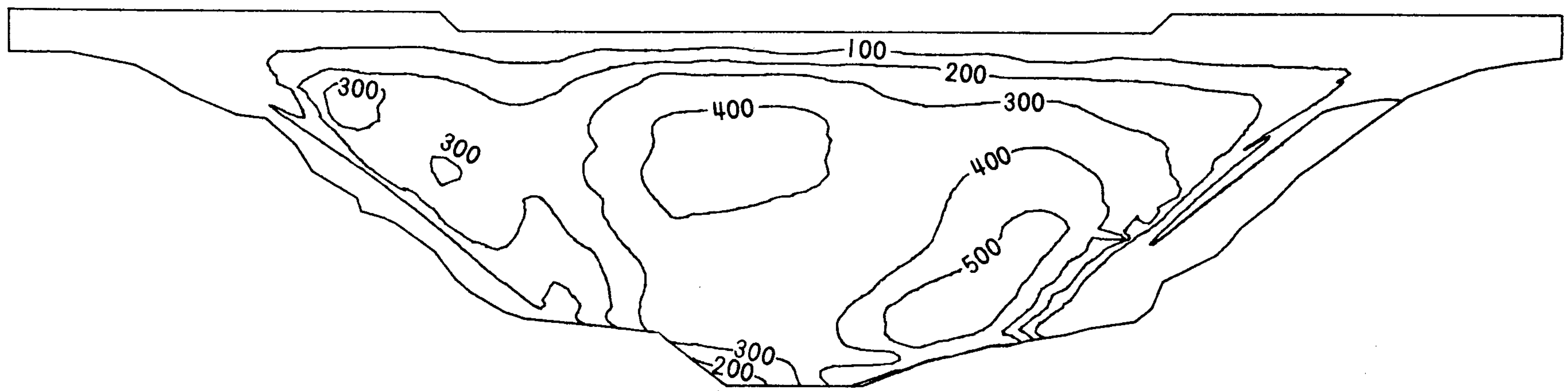


FIGURE 9-22 ENVELOPE VALUES OF MAXIMUM (TENSILE) CANTILEVER STRESSES ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

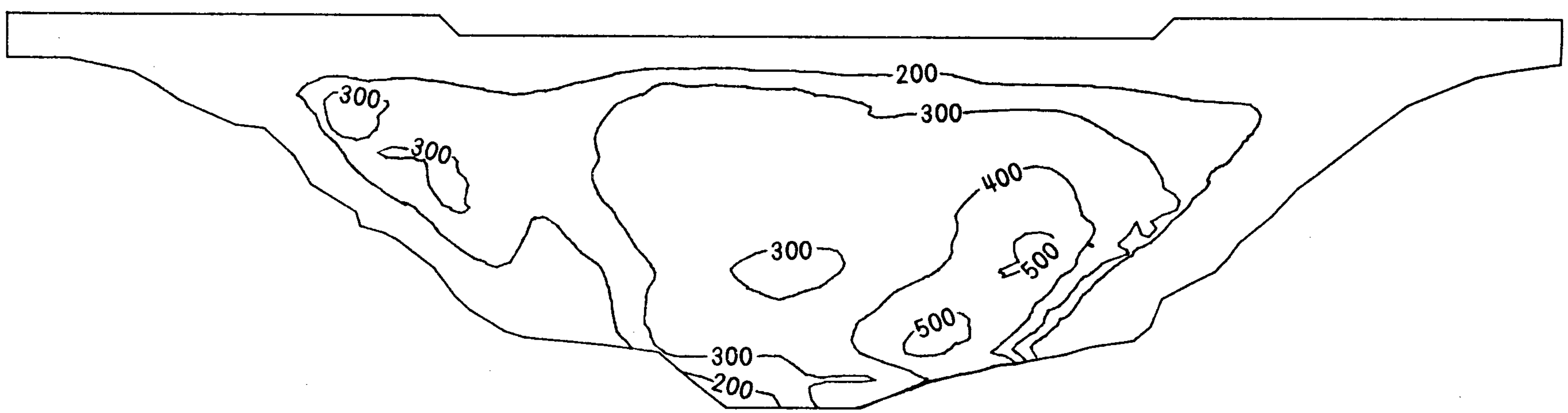
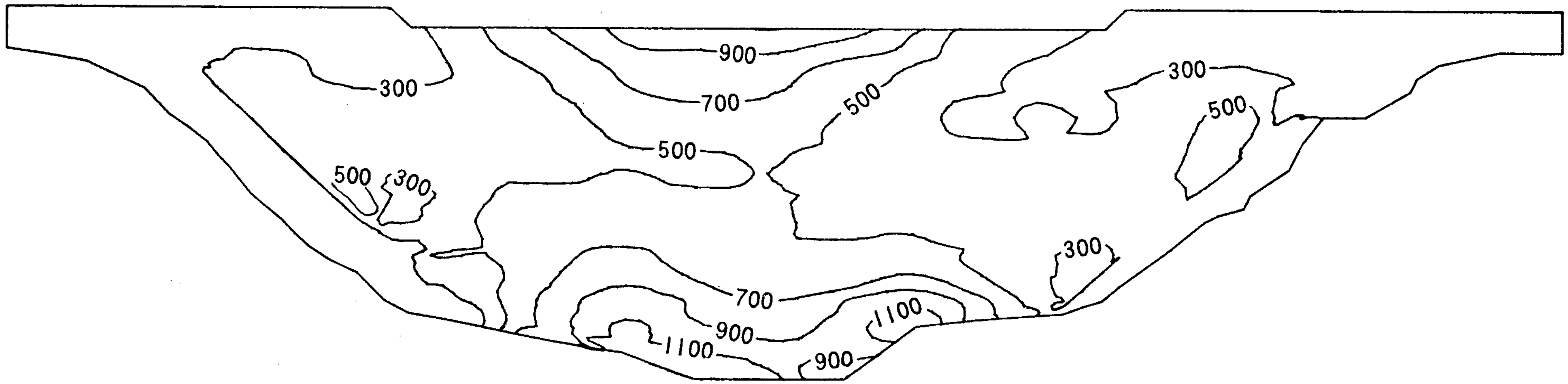


FIGURE 9-23 ENVELOPE VALUES OF MAXIMUM (TENSILE) CANTILEVER STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

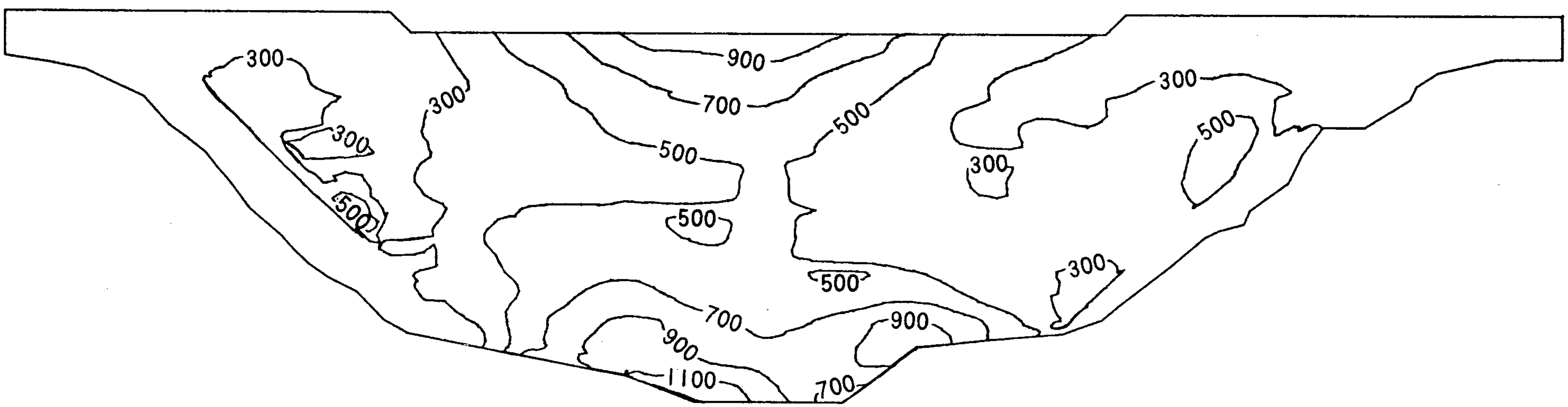
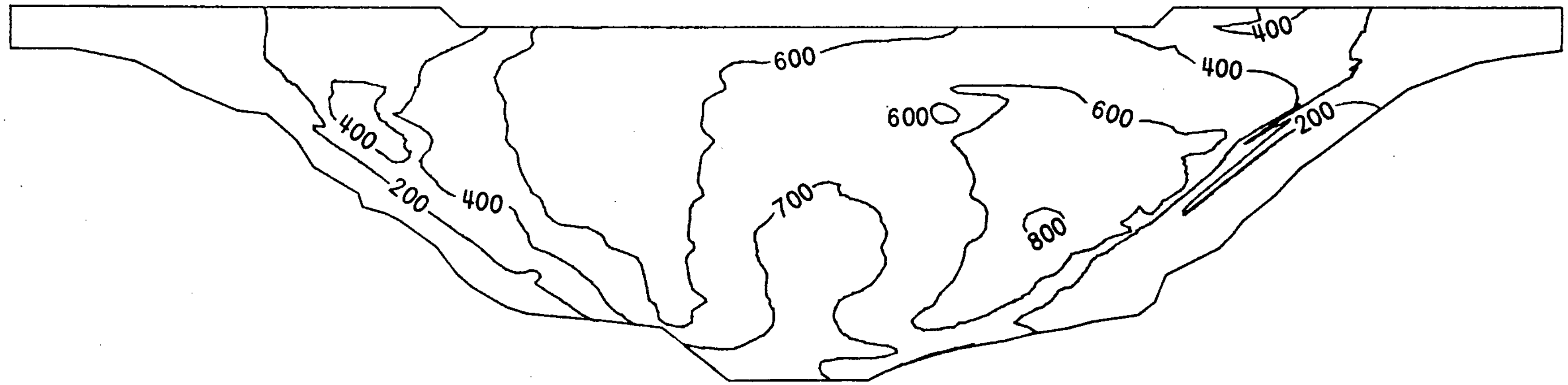


FIGURE 9-24 ENVELOPE VALUES OF MAJOR (TENSILE) PRINCIPAL STRESSES ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

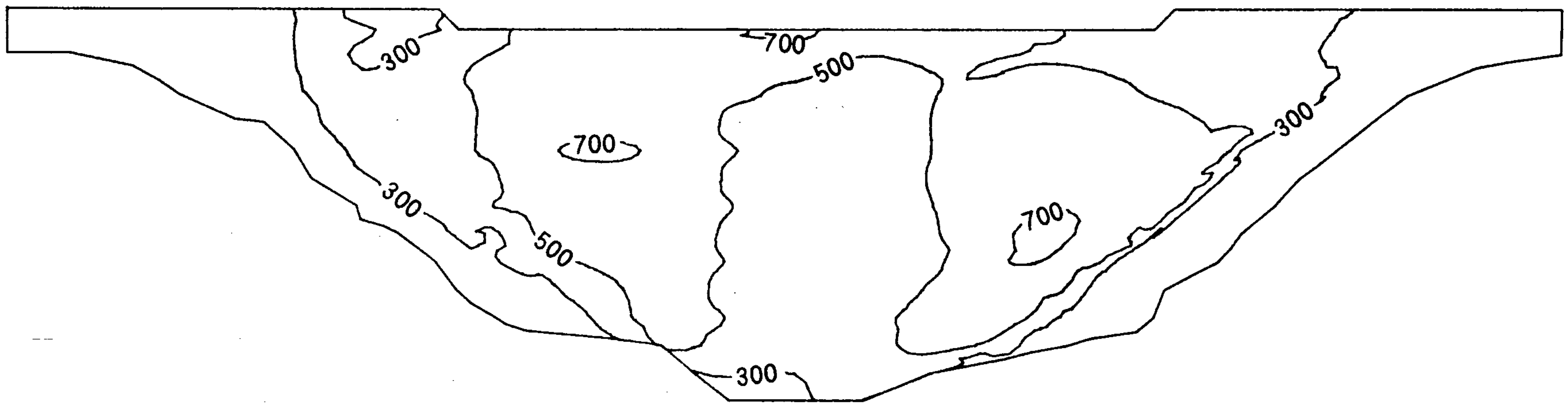
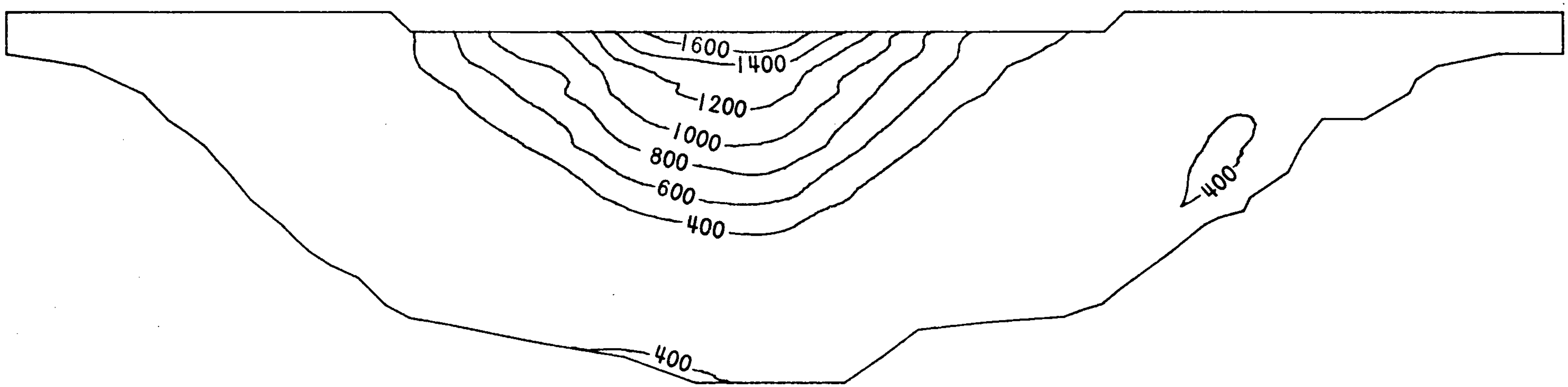


FIGURE 9-25 ENVELOPE VALUES OF **MAJOR** (TENSILE) PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

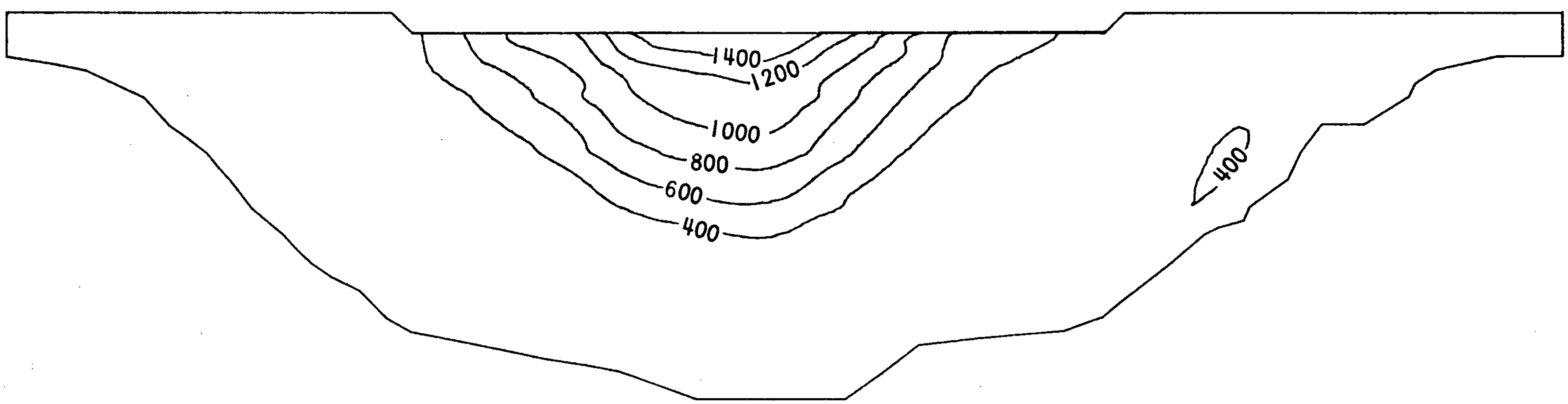
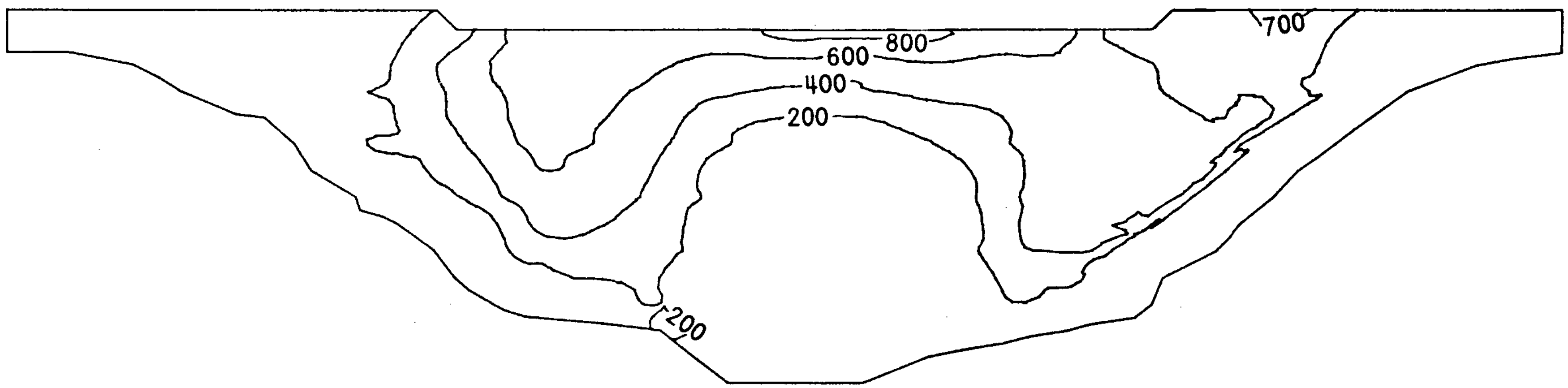


FIGURE 9-26 ENVELOPE VALUES OF MAXIMUM (TENSILE) ARCH STRESSES ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

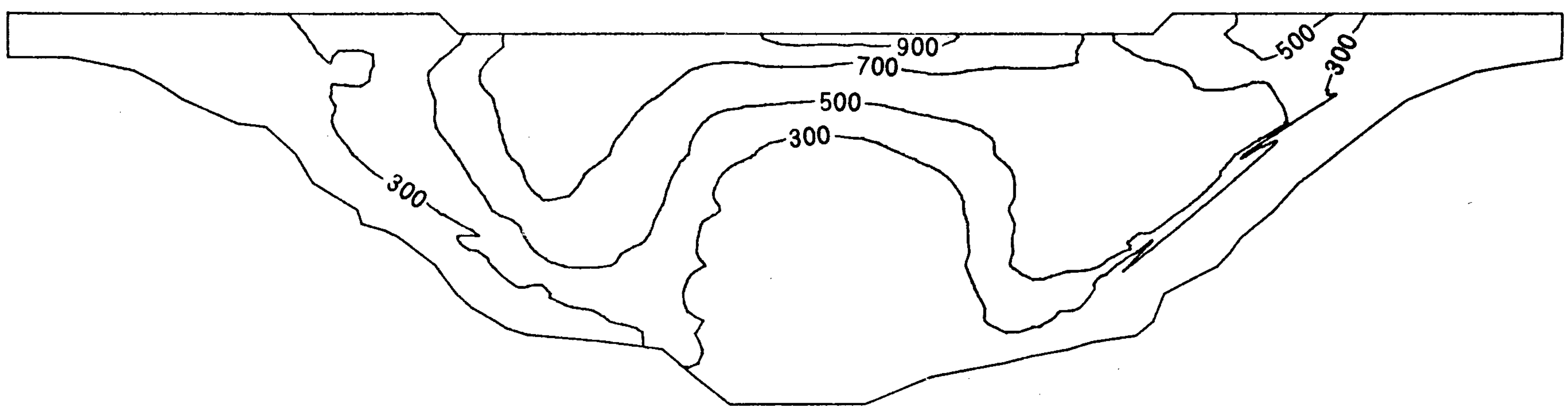


FIGURE 9-27 ENVELOPE VALUES OF MAXIMUM (TENSILE) ARCH STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90

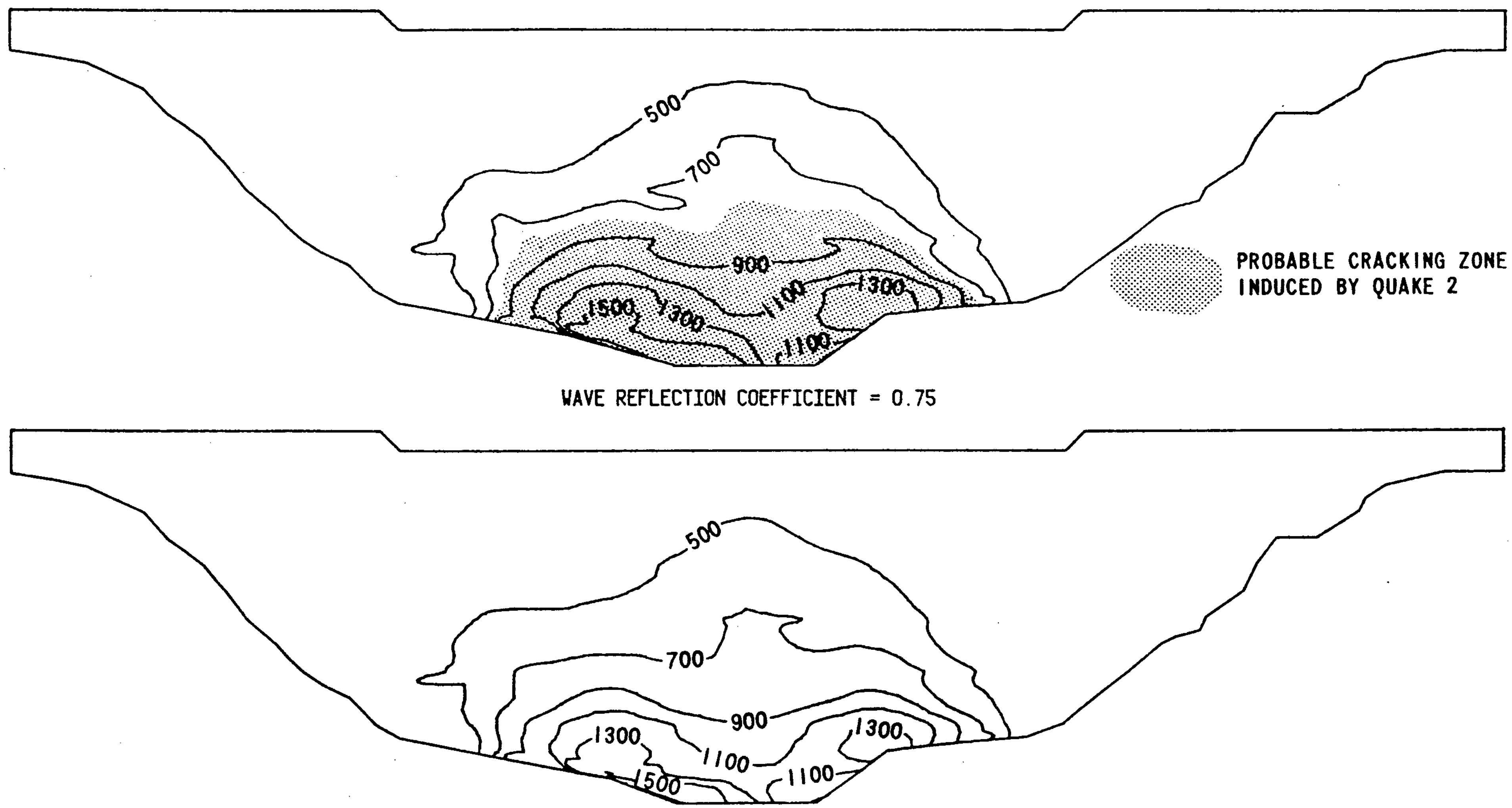


FIGURE 9-28 ENVELOPE VALUES OF MAXIMUM (TENSILE) CANTILEVER STRESSES ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

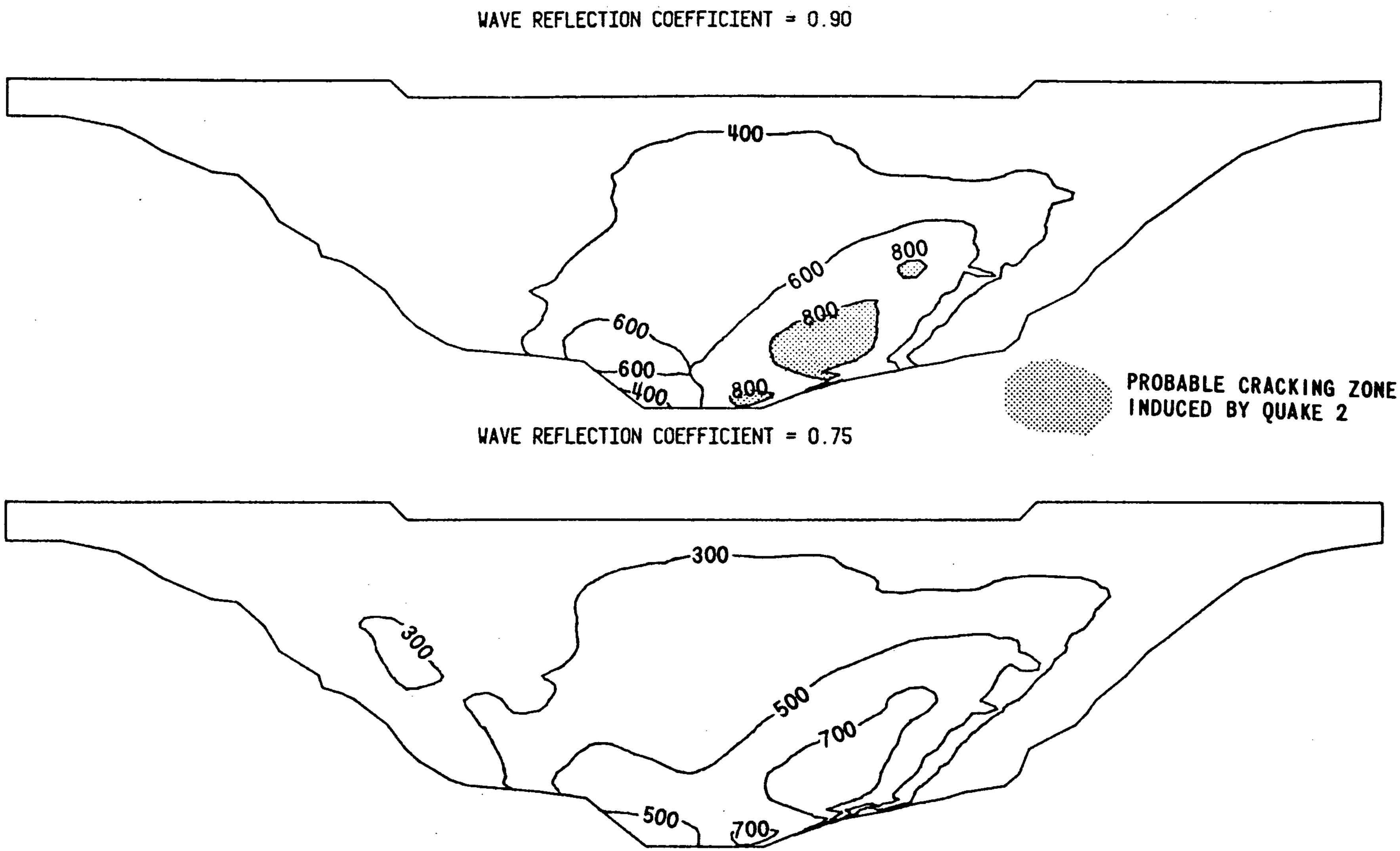
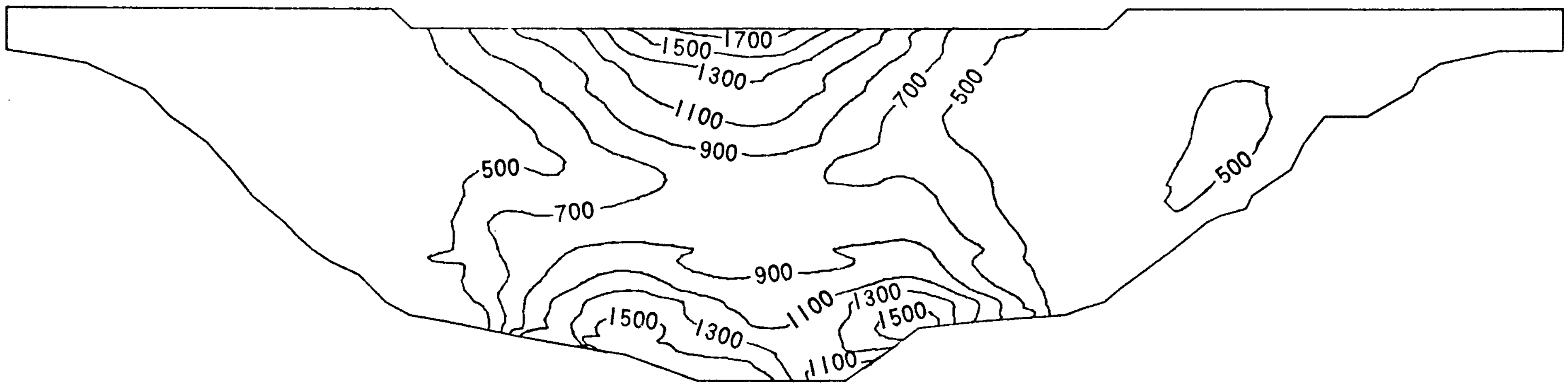


FIGURE 9-29 ENVELOPE VALUES OF MAXIMUM (TENSILE) CANTILEVER STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

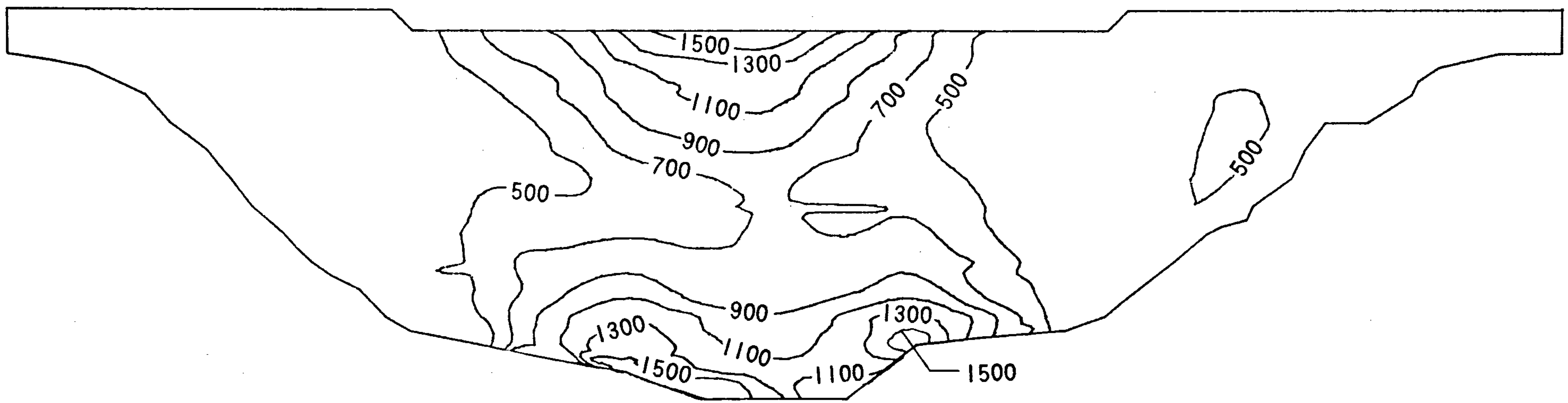


FIGURE 9-30 ENVELOPE VALUES OF MAJOR (TENSILE) PRINCIPAL STRESSES ON UPSTREAM FACE OF THE DAM UN FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

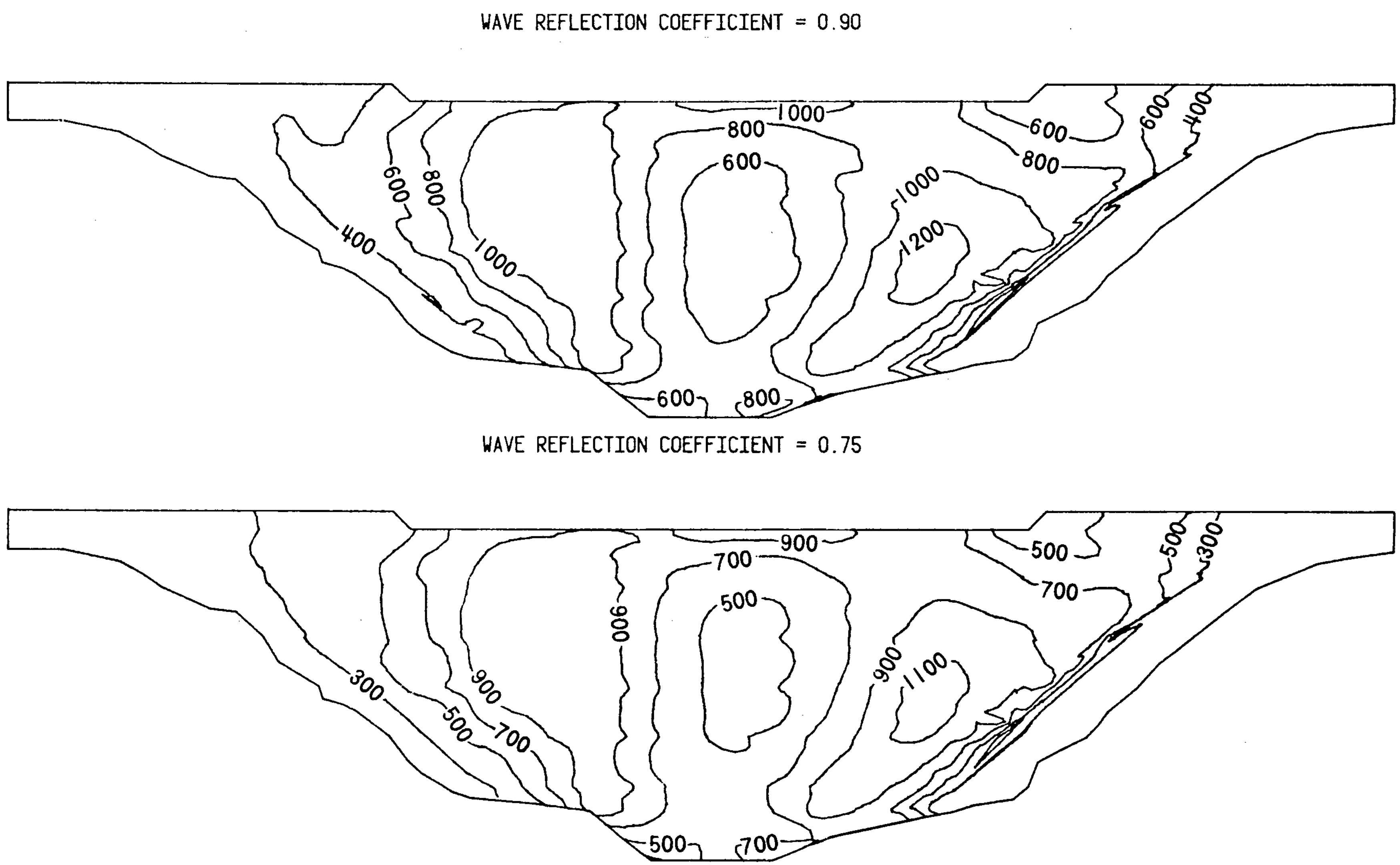


FIGURE 9-31 ENVELOPE VALUES OF **MAJOR** (TENSILE) PRINCIPAL STRESSES ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

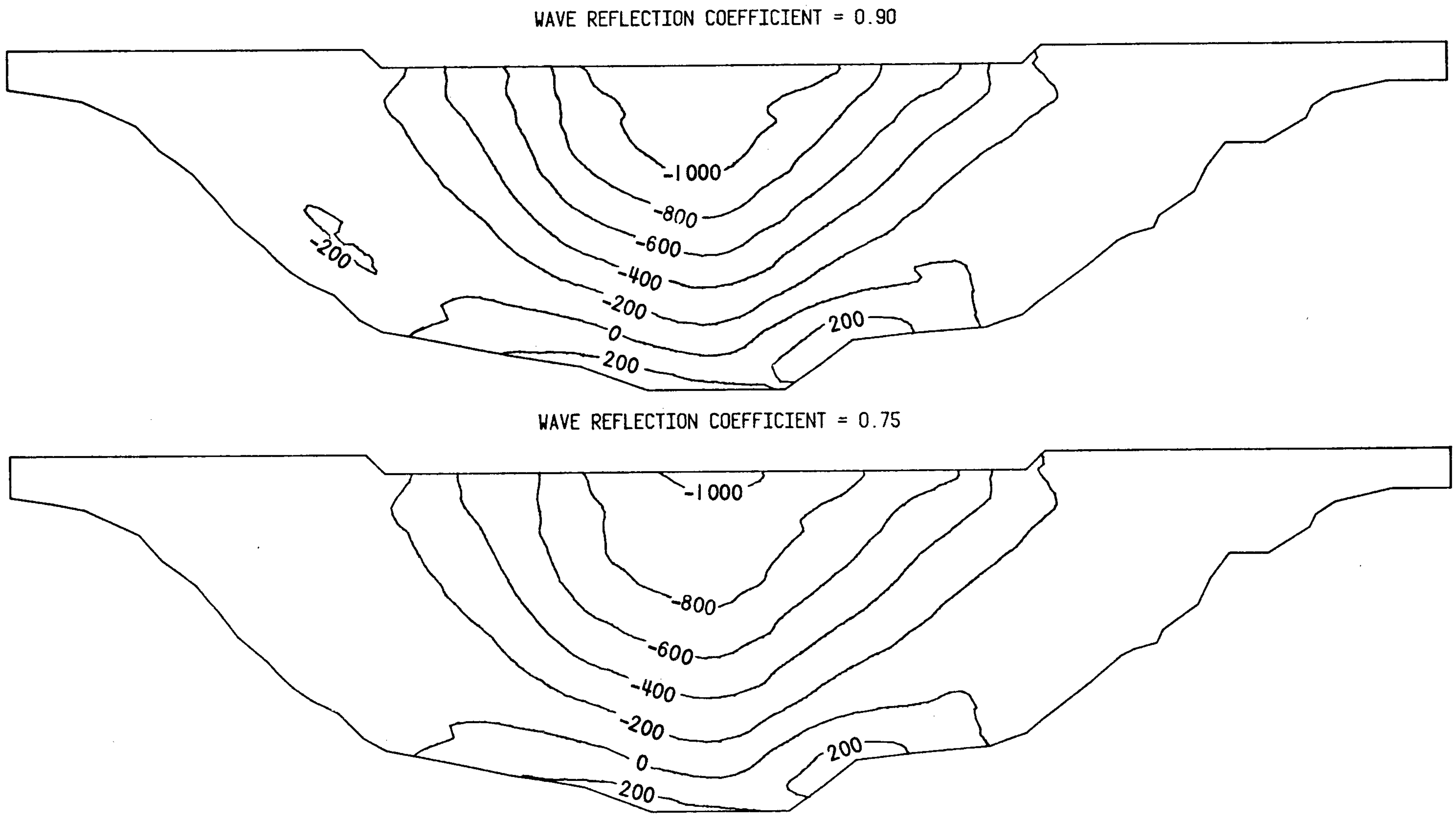
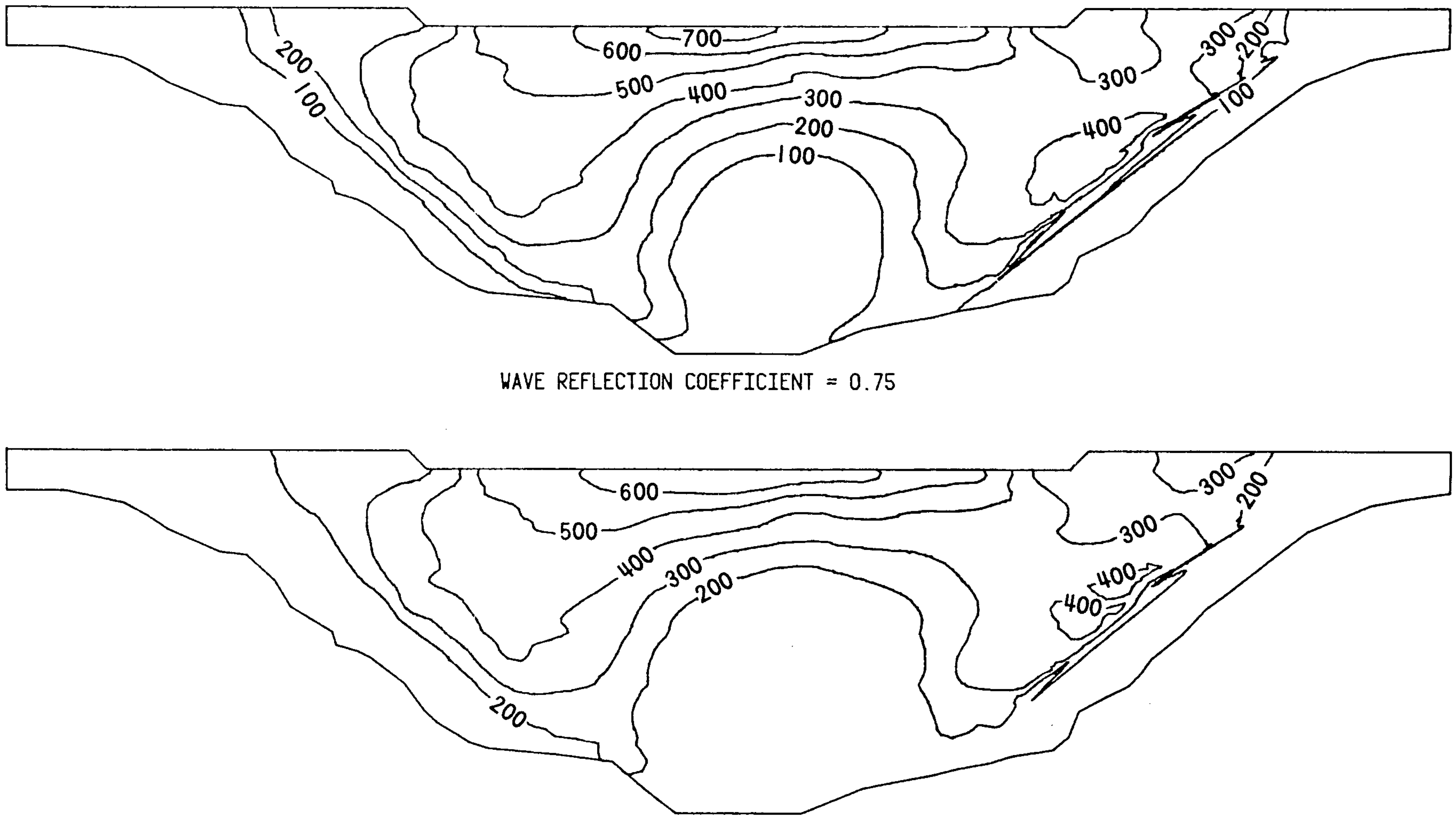


FIGURE 9-32 INSTANTANEOUS VALUES OF ARCH STRESSES (AT 4.92 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

FIGURE 9-33 INSTANTANEOUS VALUES OF ARCH STRESSES (AT 7.24 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

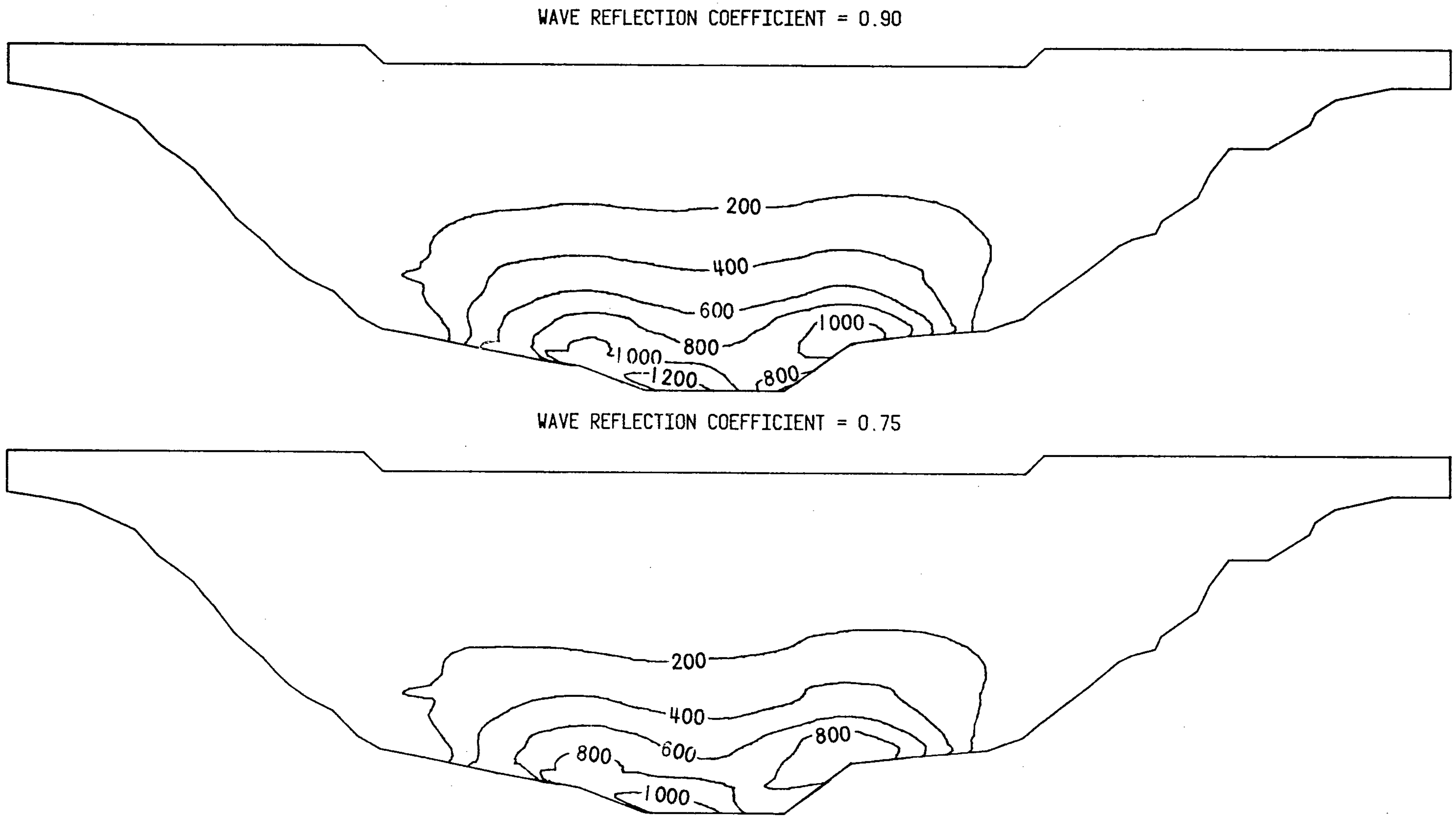
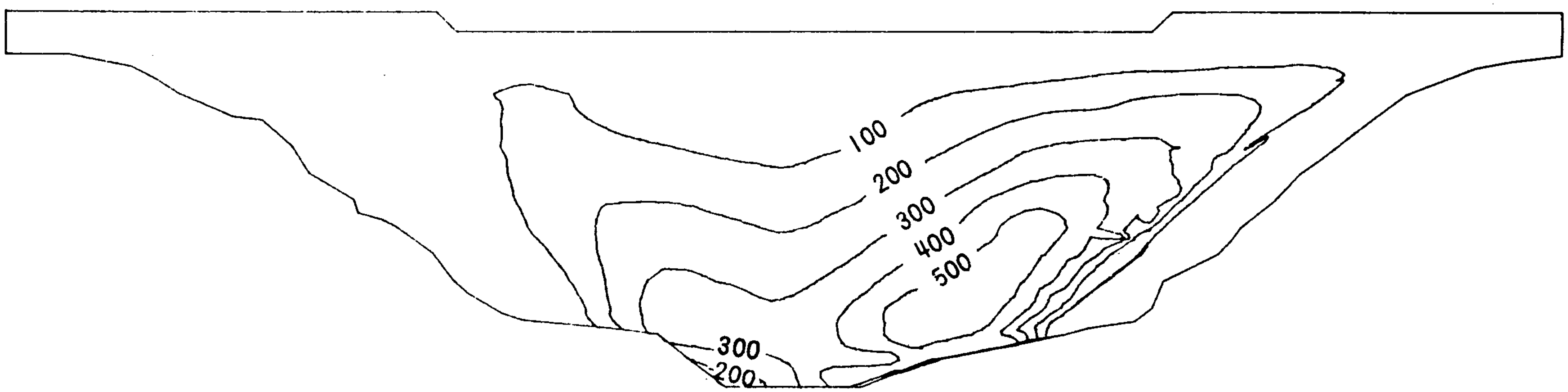


FIGURE 9-34 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.92 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

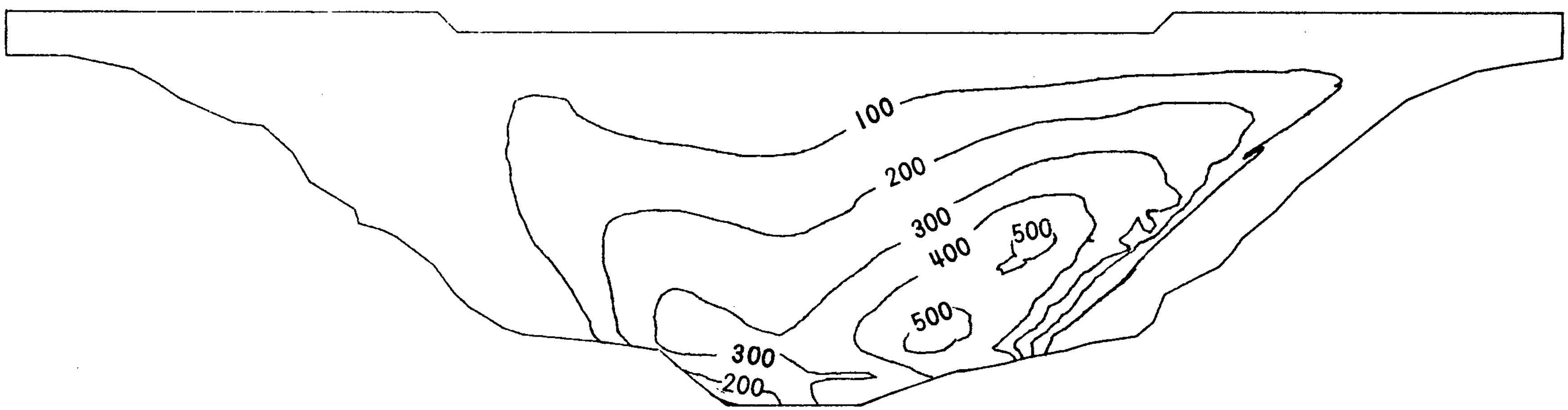


FIGURE 9-35 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 7.24 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90

WAVE REFLECTION COEFFICIENT = 0.75

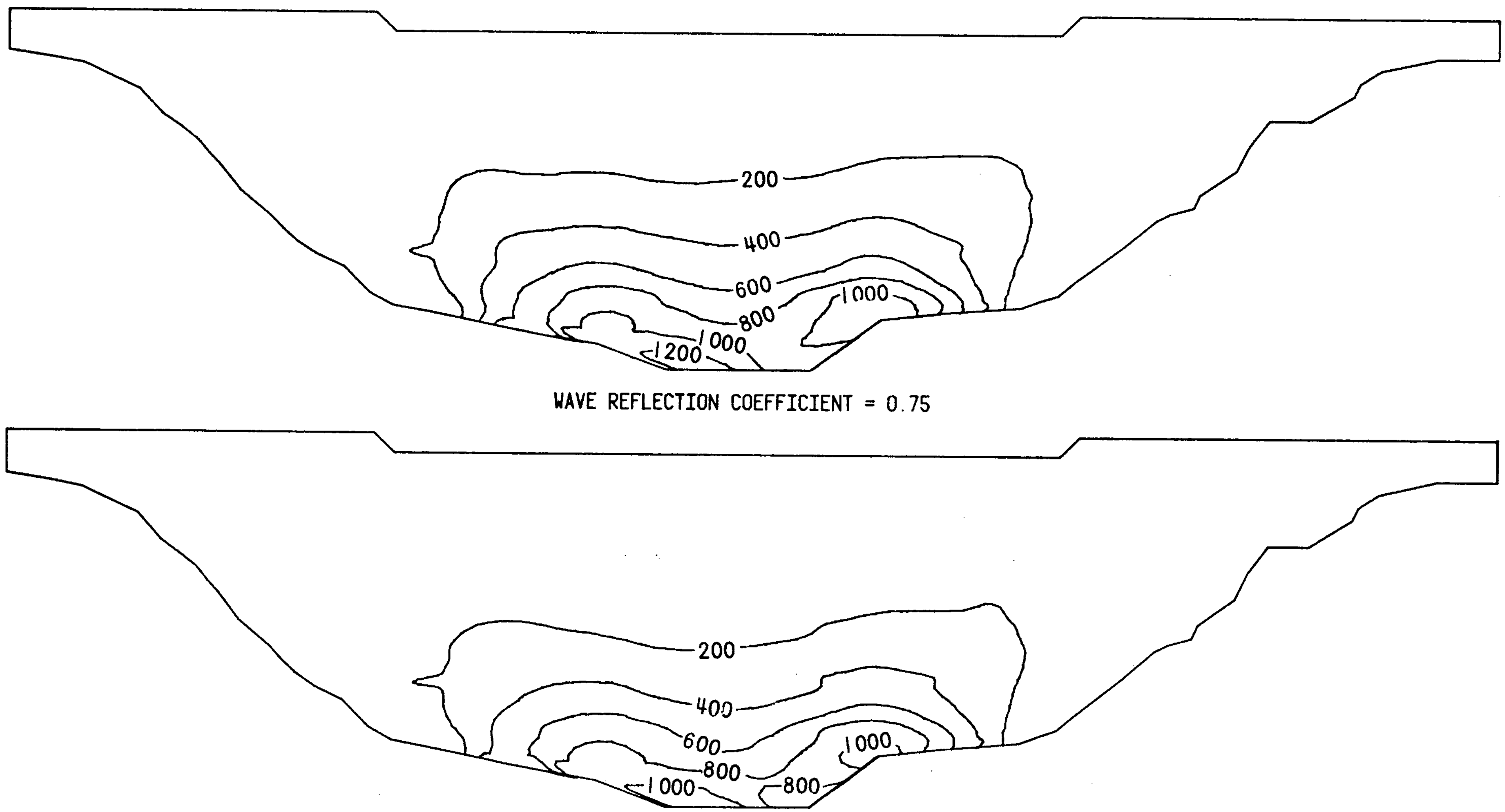
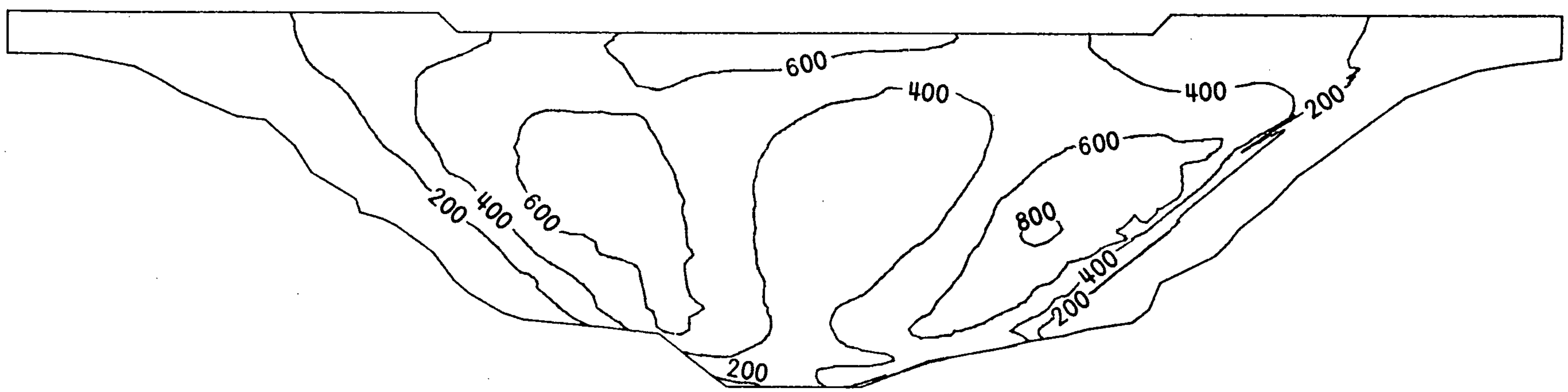


FIGURE 9-36 INSTANTANEOUS VALUES OF MAJOR PRINCIPAL STRESSES (AT 4.92 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

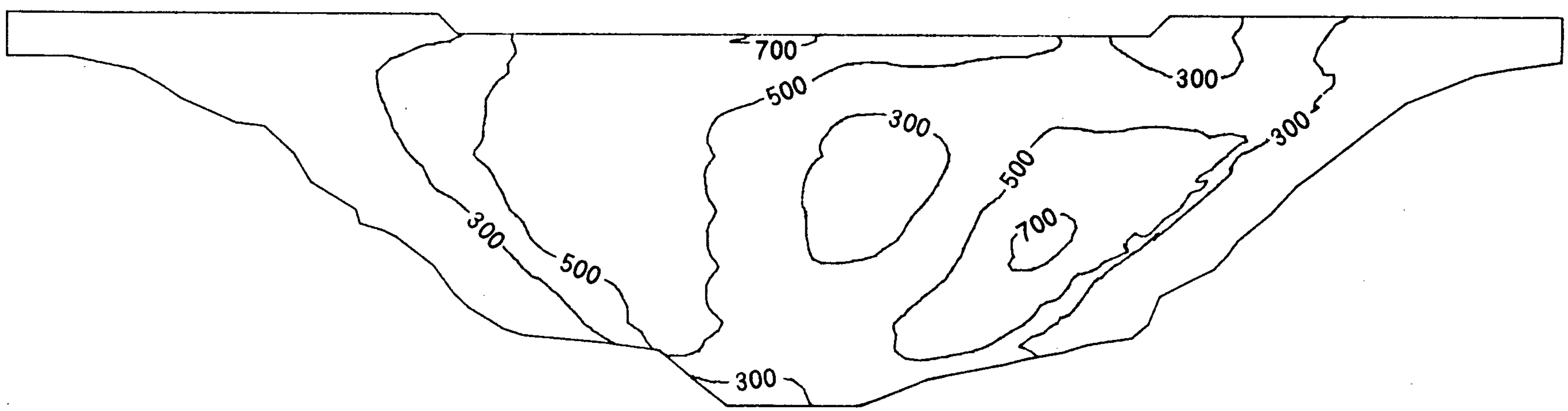
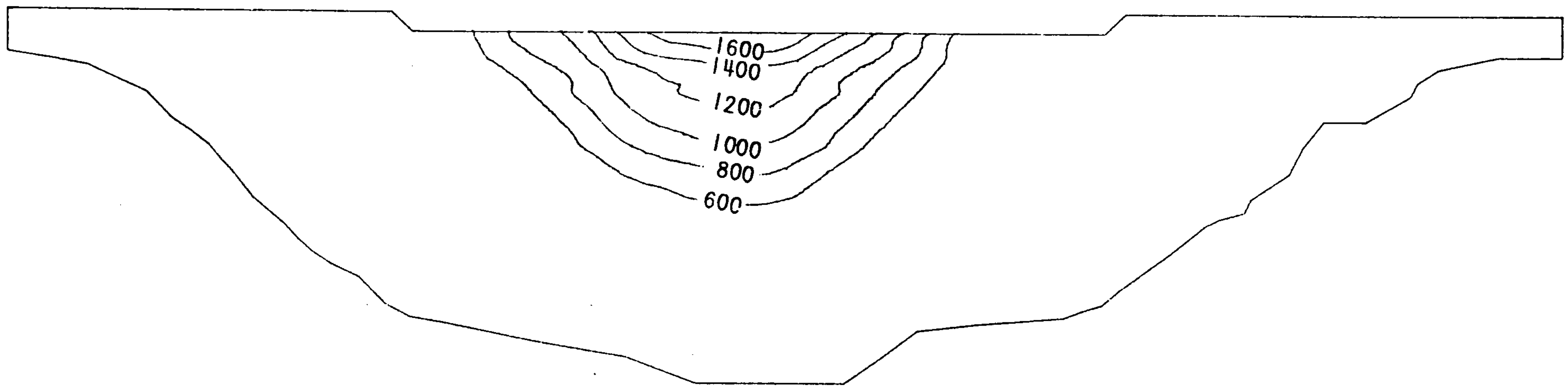


FIGURE 9-37 INSTANTANEOUS VALUES OF MAJOR PRINCIPAL STRESSES (AT 7.24 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

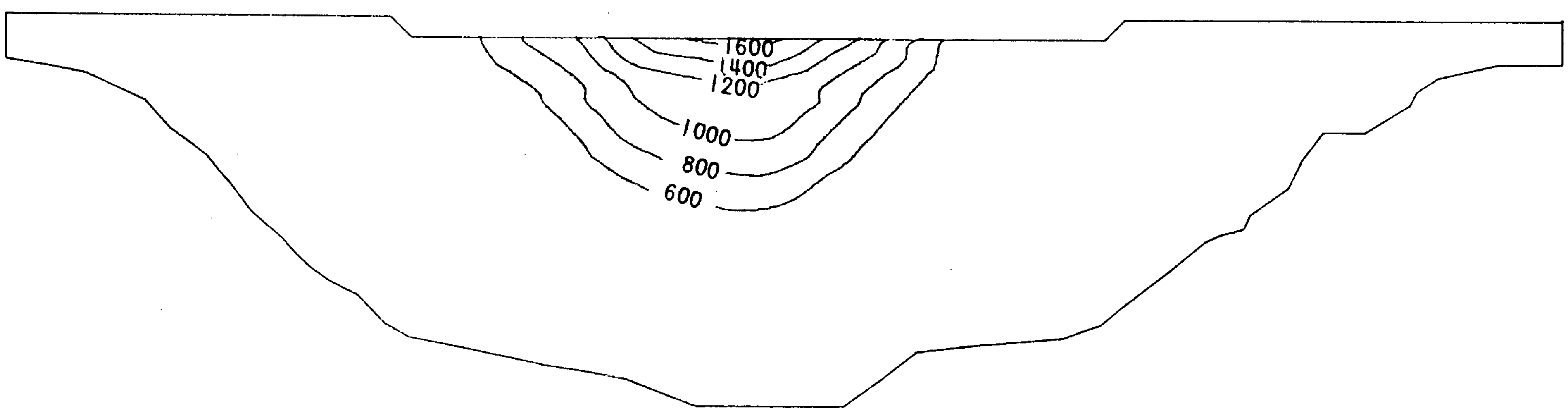
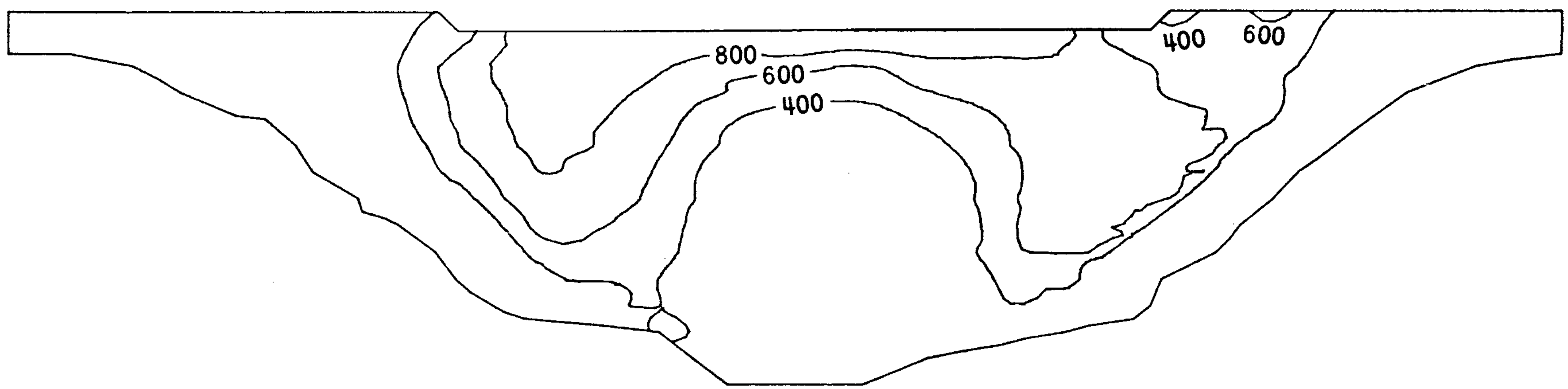


FIGURE 9-38 INSTANTANEOUS VALUES OF ARCH STRESSES (AT 4.66 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

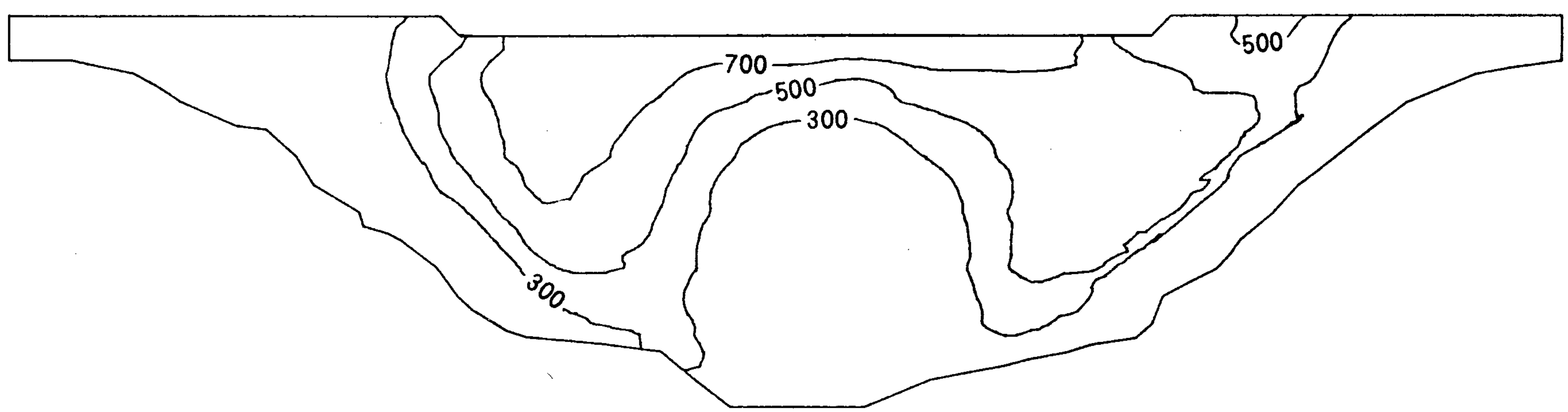
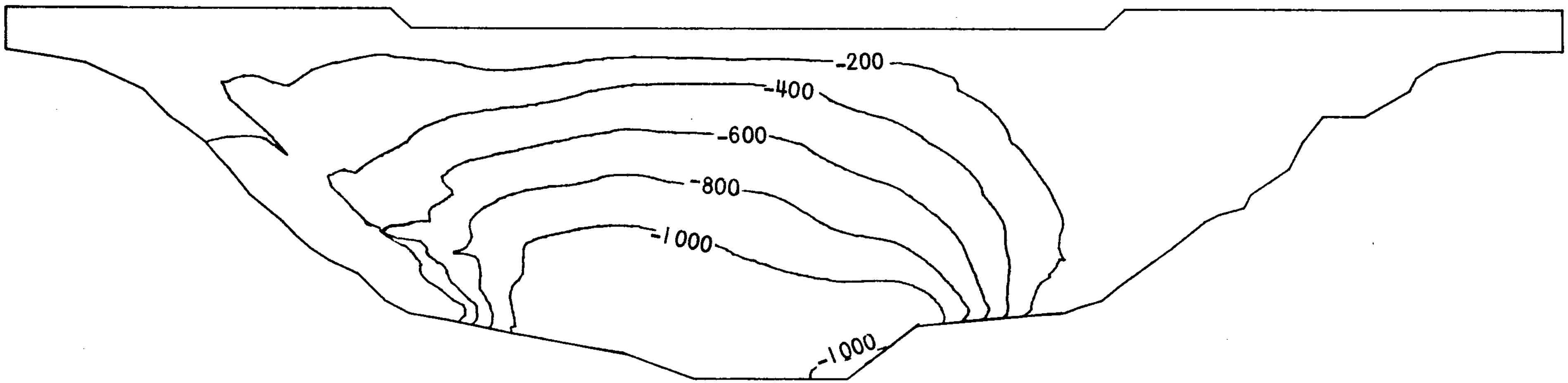


FIGURE 9-39 INSTANTANEOUS VALUES OF ARCH STRESSES (AT 4.66 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

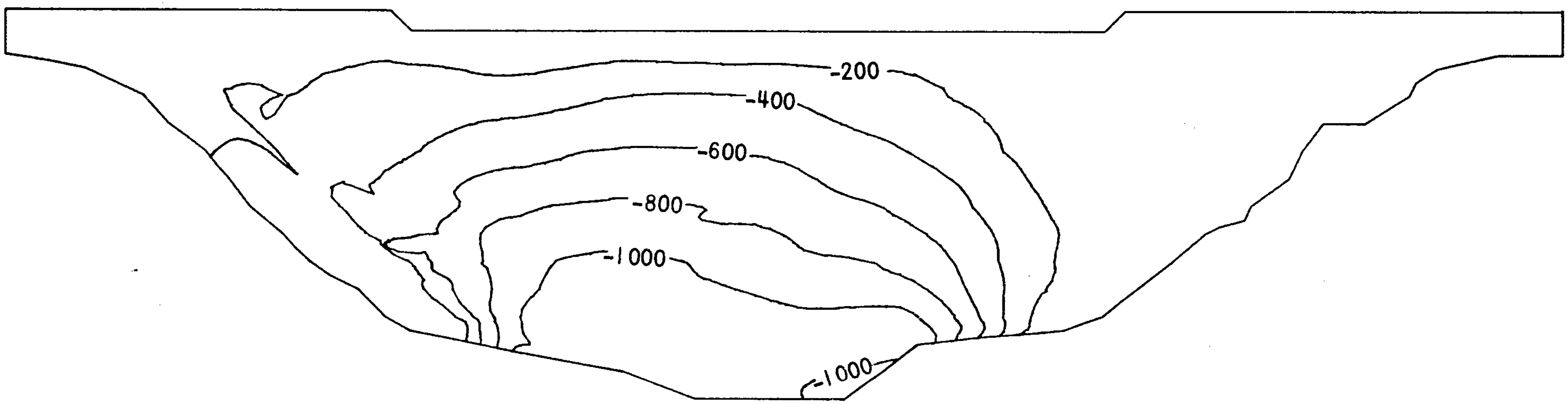
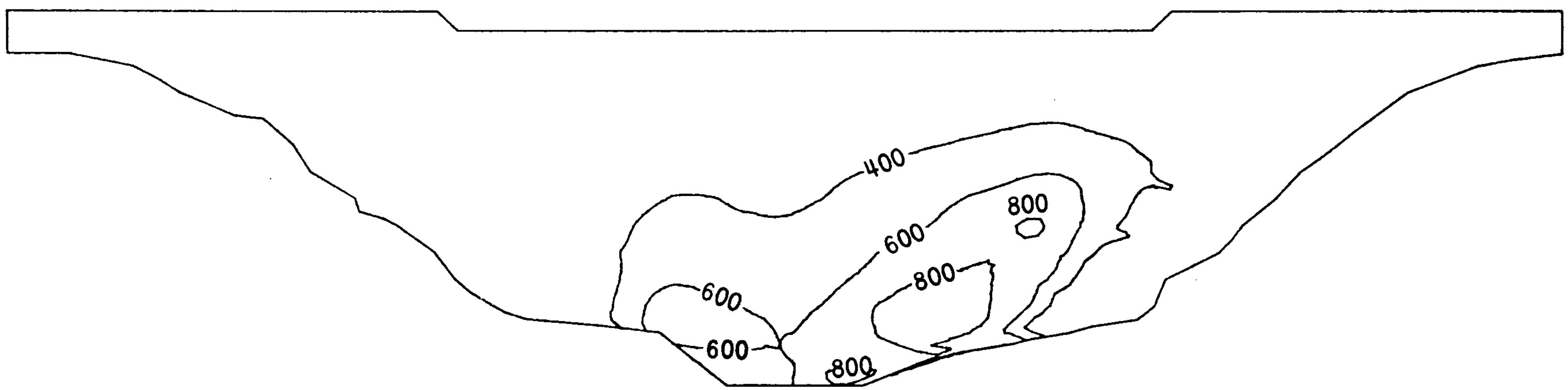


FIGURE 9-40 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.66 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

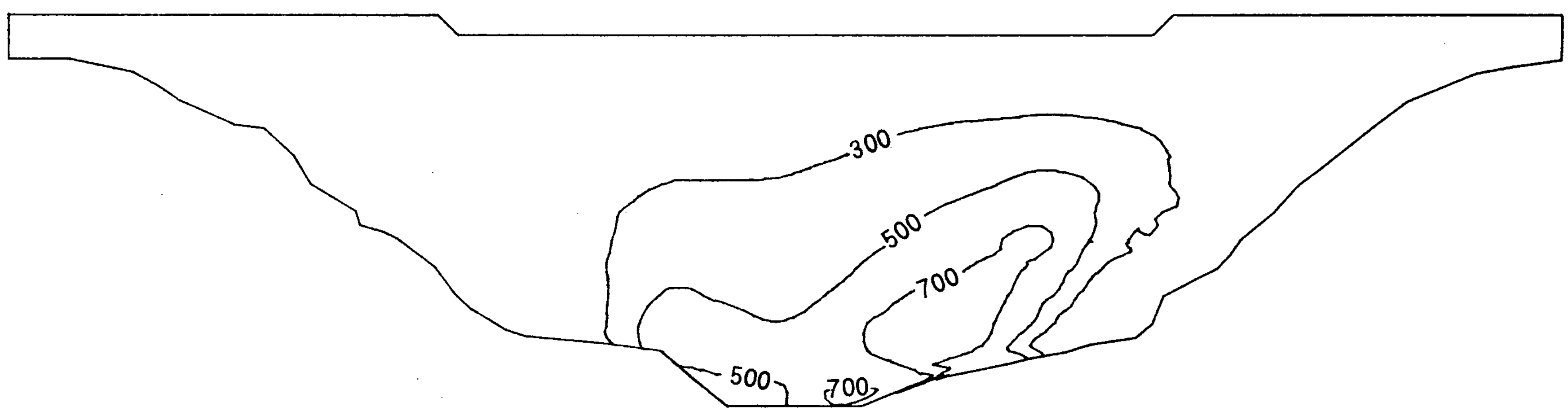


FIGURE 9-41 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.66 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

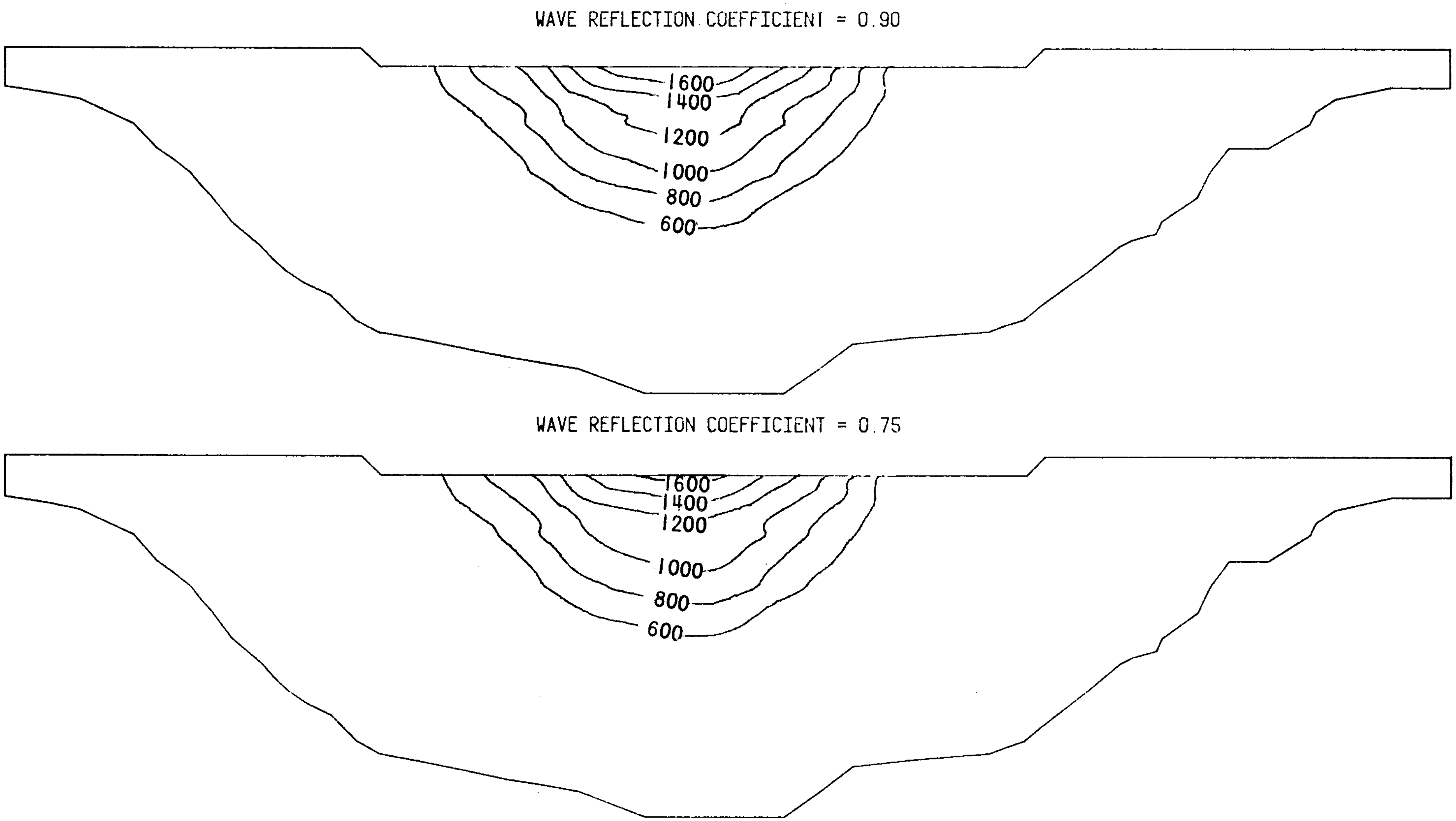
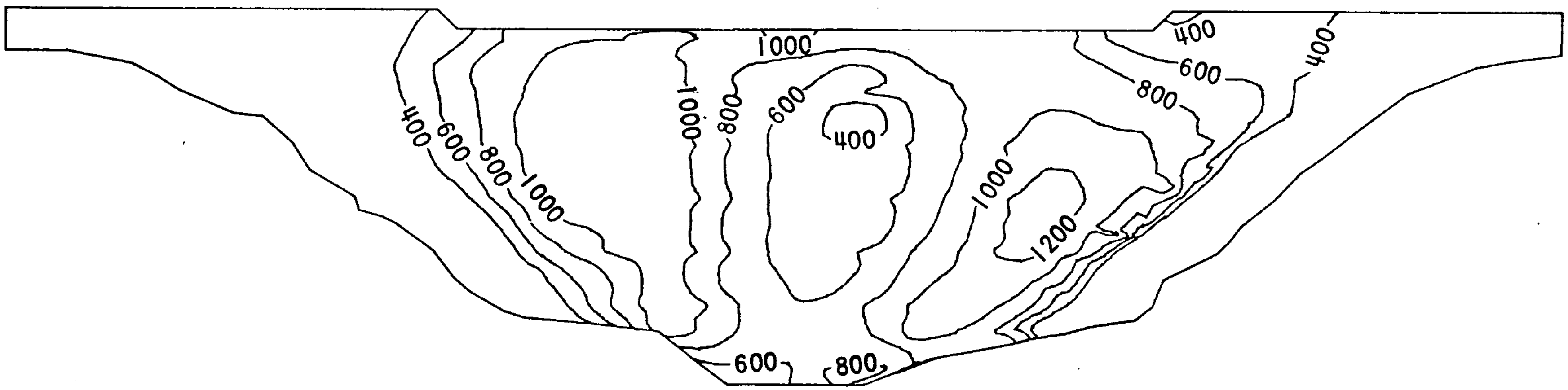


FIGURE 9-42 INSTANTANEOUS VALUES OF MAJOR PRINCIPAL STRESSES (AT 4.66 SEC.) ON UPSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

WAVE REFLECTION COEFFICIENT = 0.90



WAVE REFLECTION COEFFICIENT = 0.75

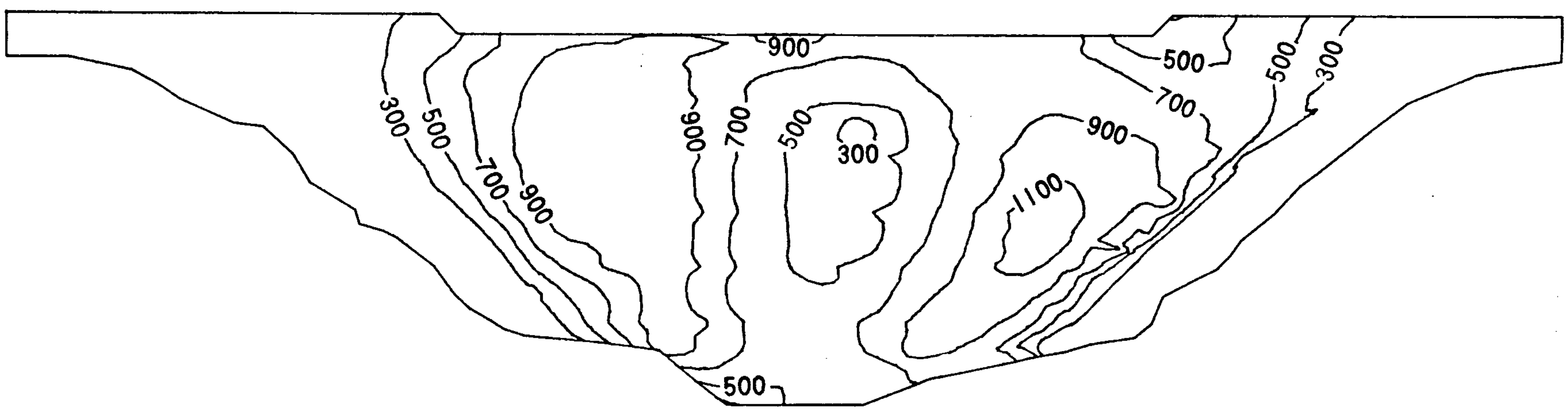


FIGURE 9-43 INSTANTANEOUS VALUES OF MAJOR PRINCIPAL STRESSES (AT 4.66 SEC.) ON DOWNSTREAM FACE OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED.

NOTE: STRESSES ARE IN P.S.I.

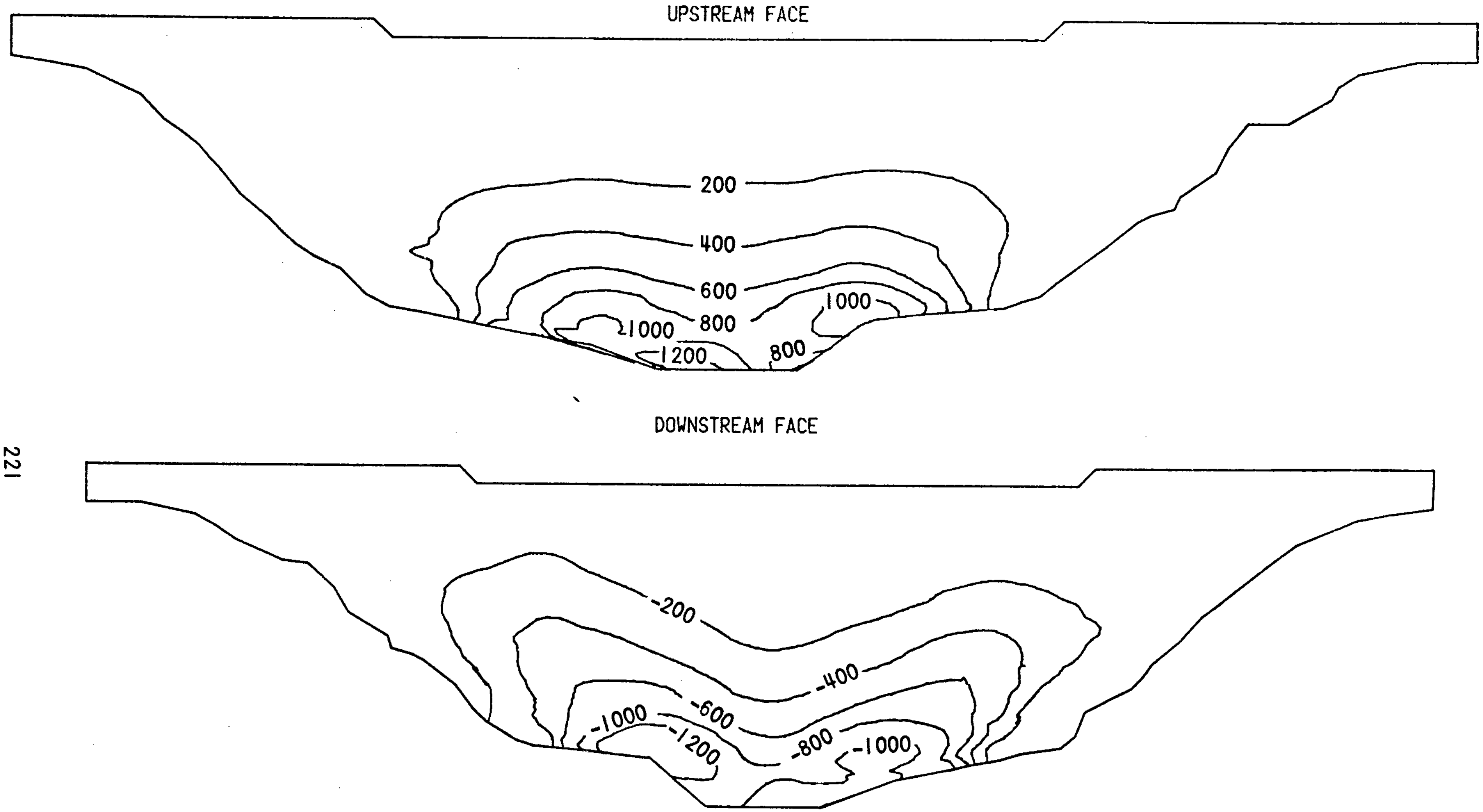


FIGURE 9-44 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.92 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

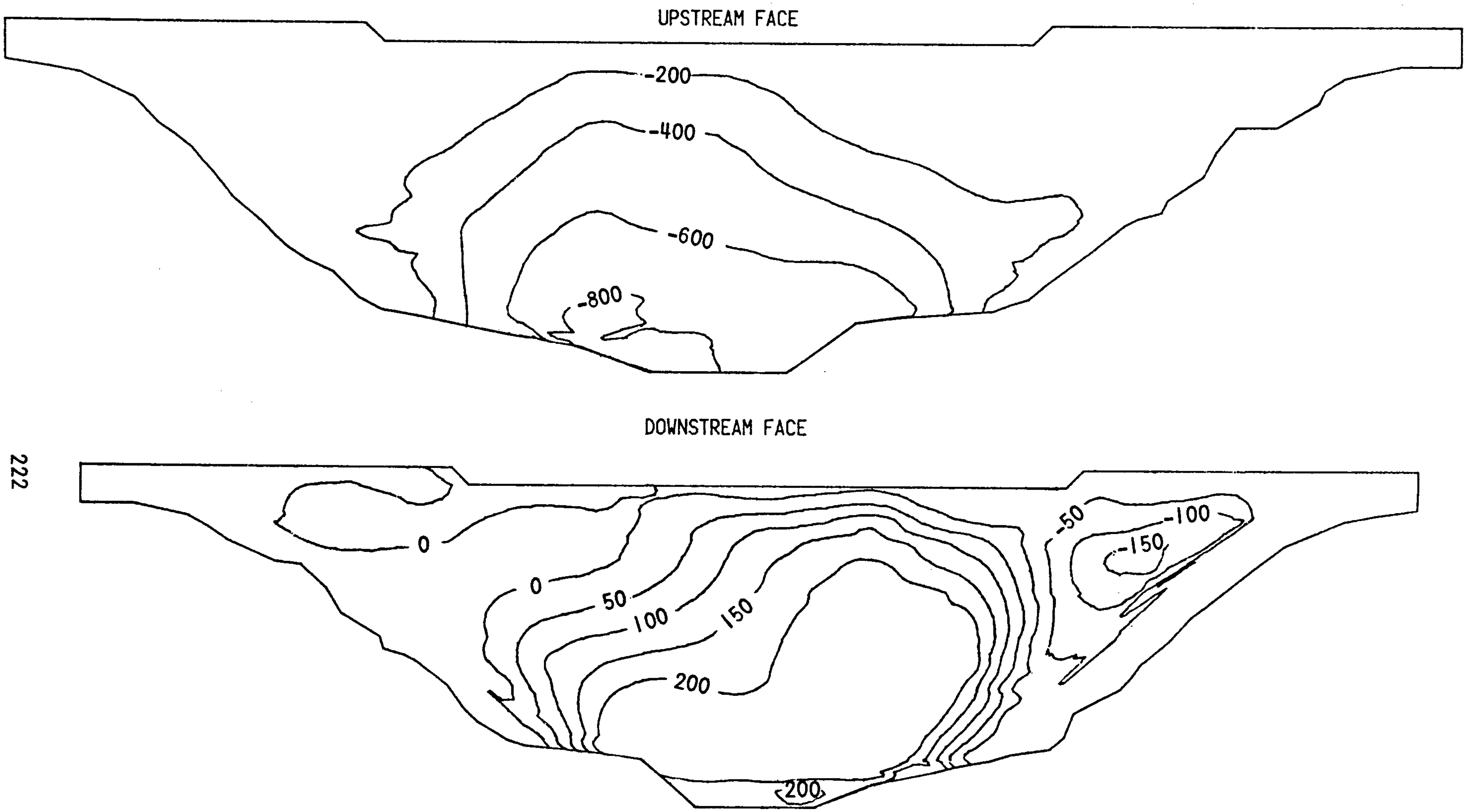


FIGURE 9-45 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 6.78 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

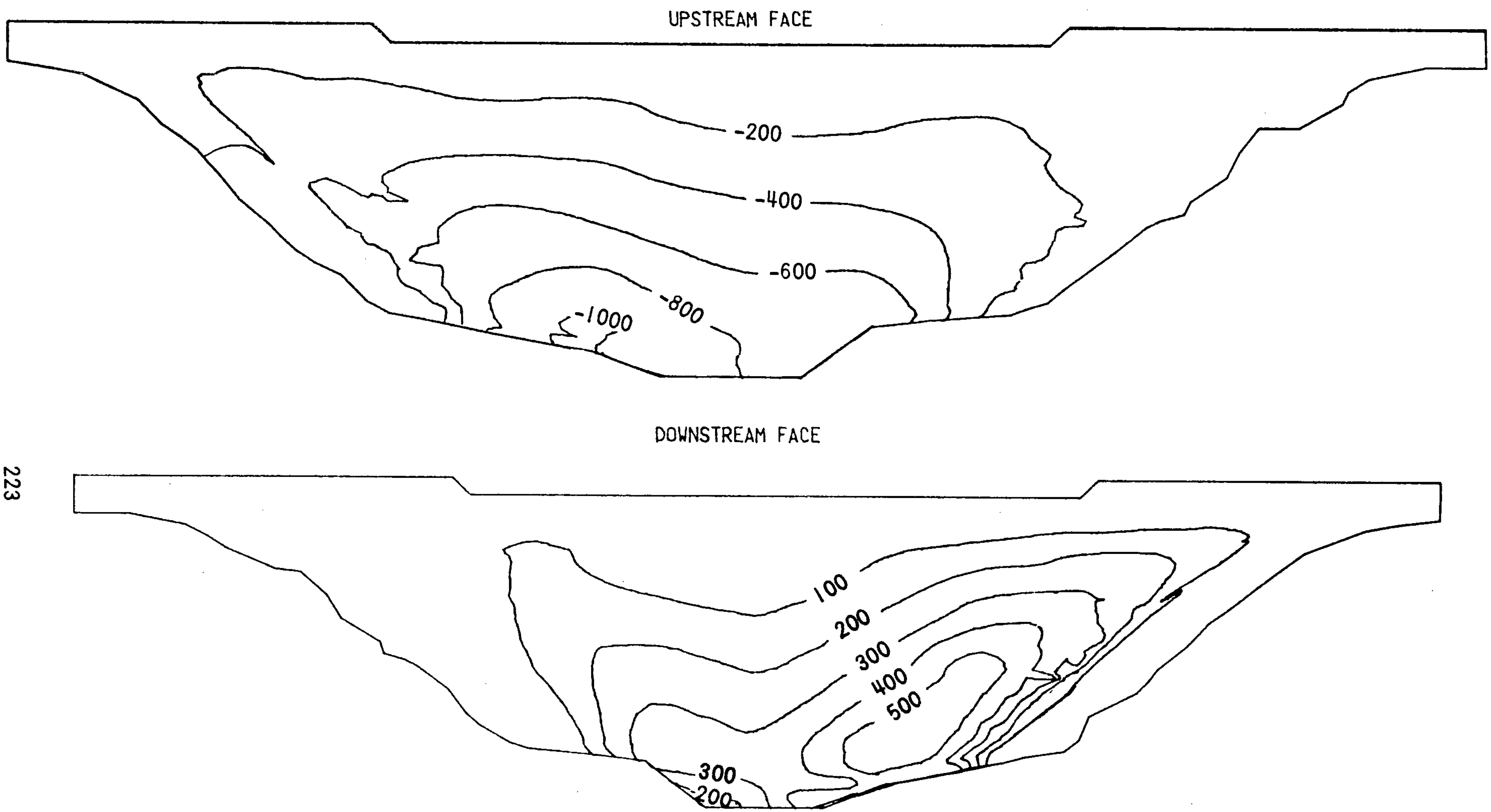


FIGURE 9-46 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 7.24 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE I. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

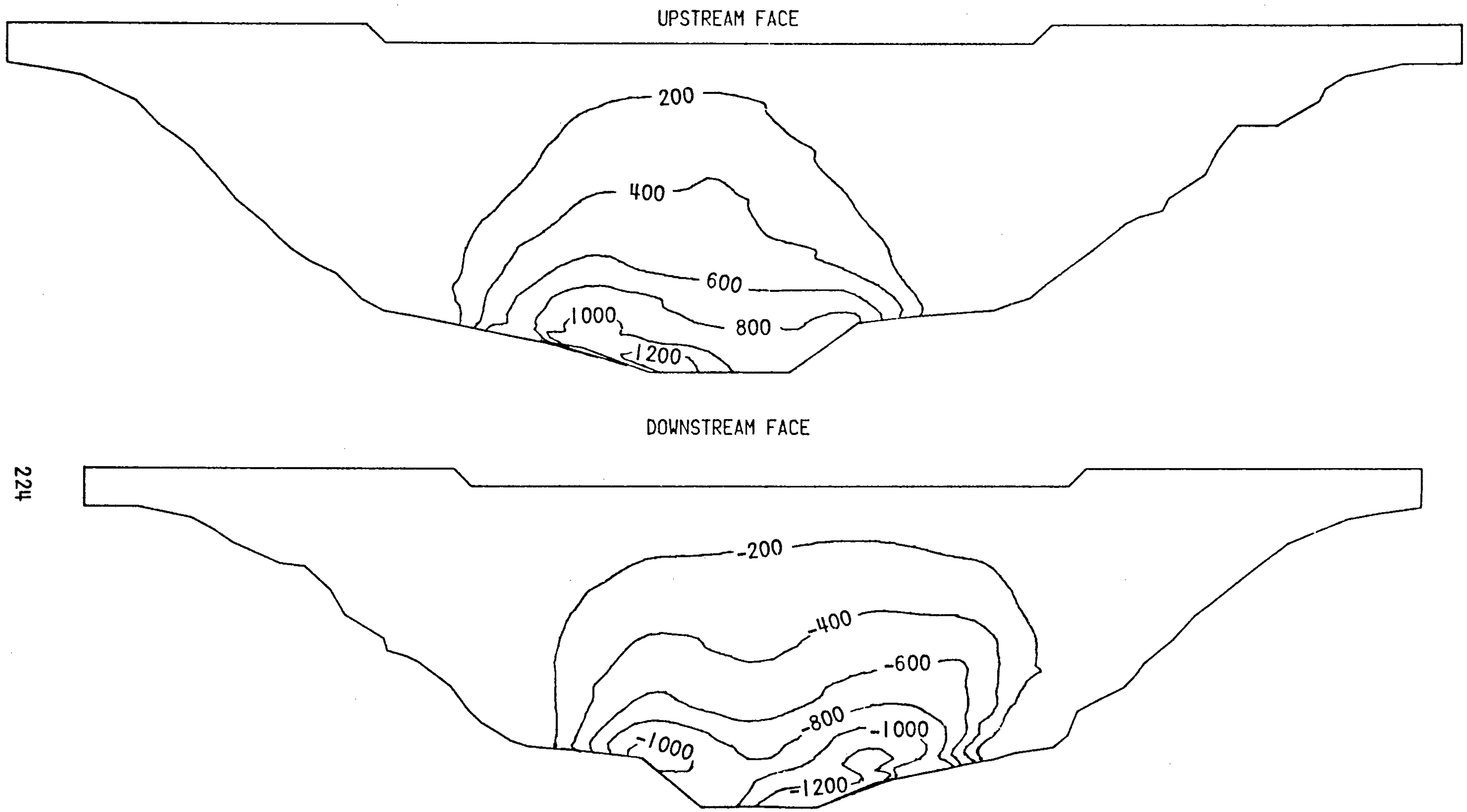


FIGURE 9-47 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 7.58 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 1. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

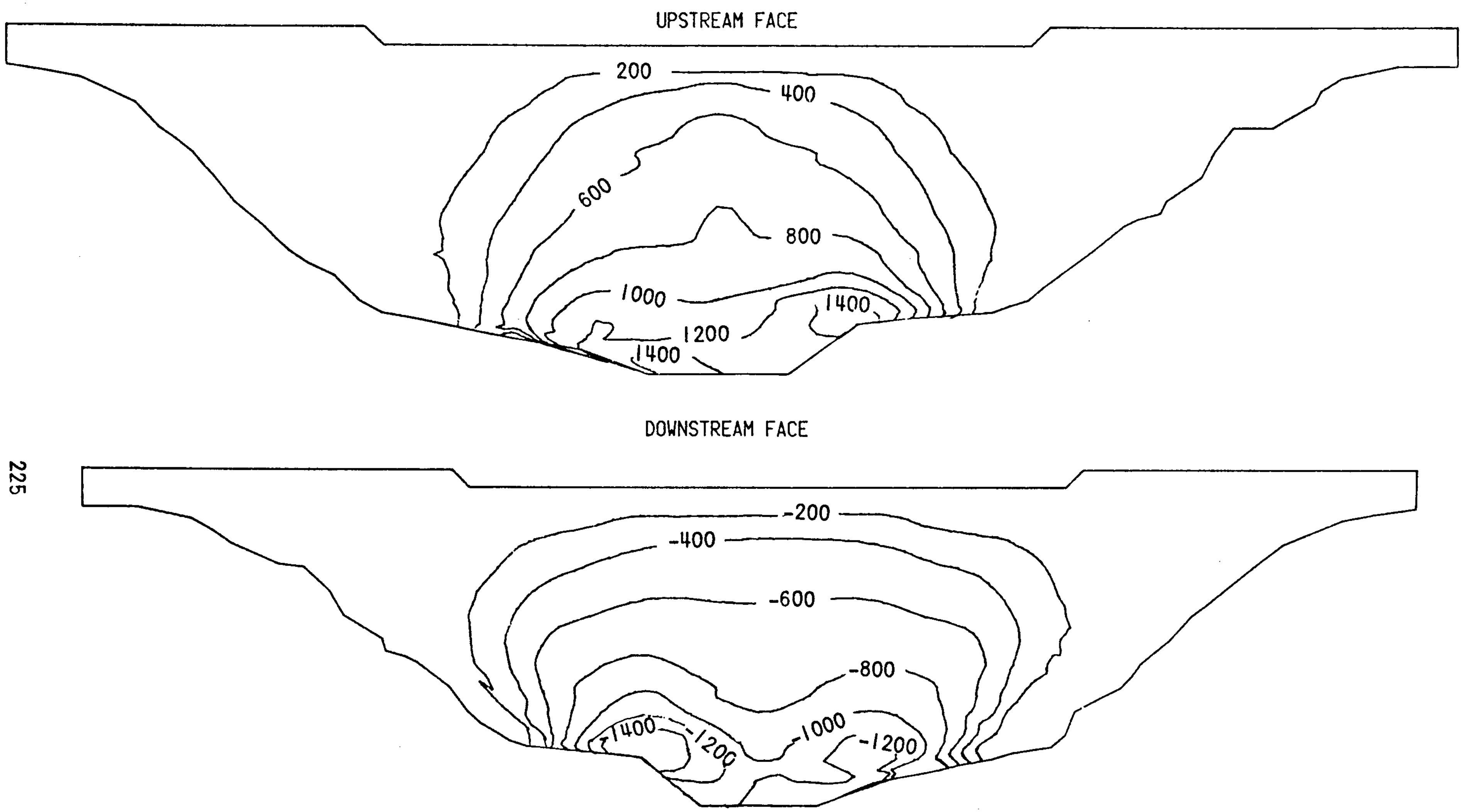


FIGURE 9-48 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 3.12 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

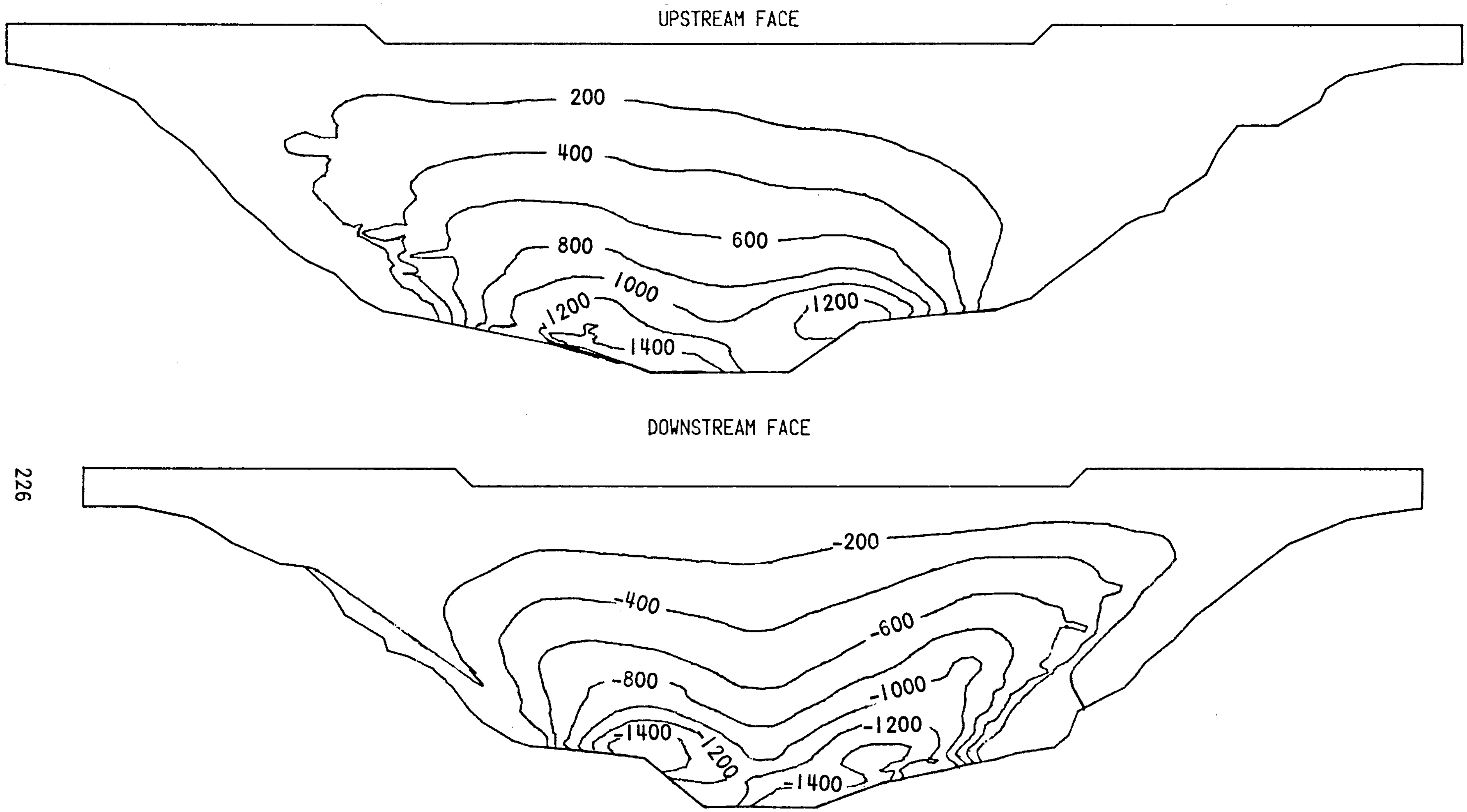


FIGURE 9-49 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.54 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

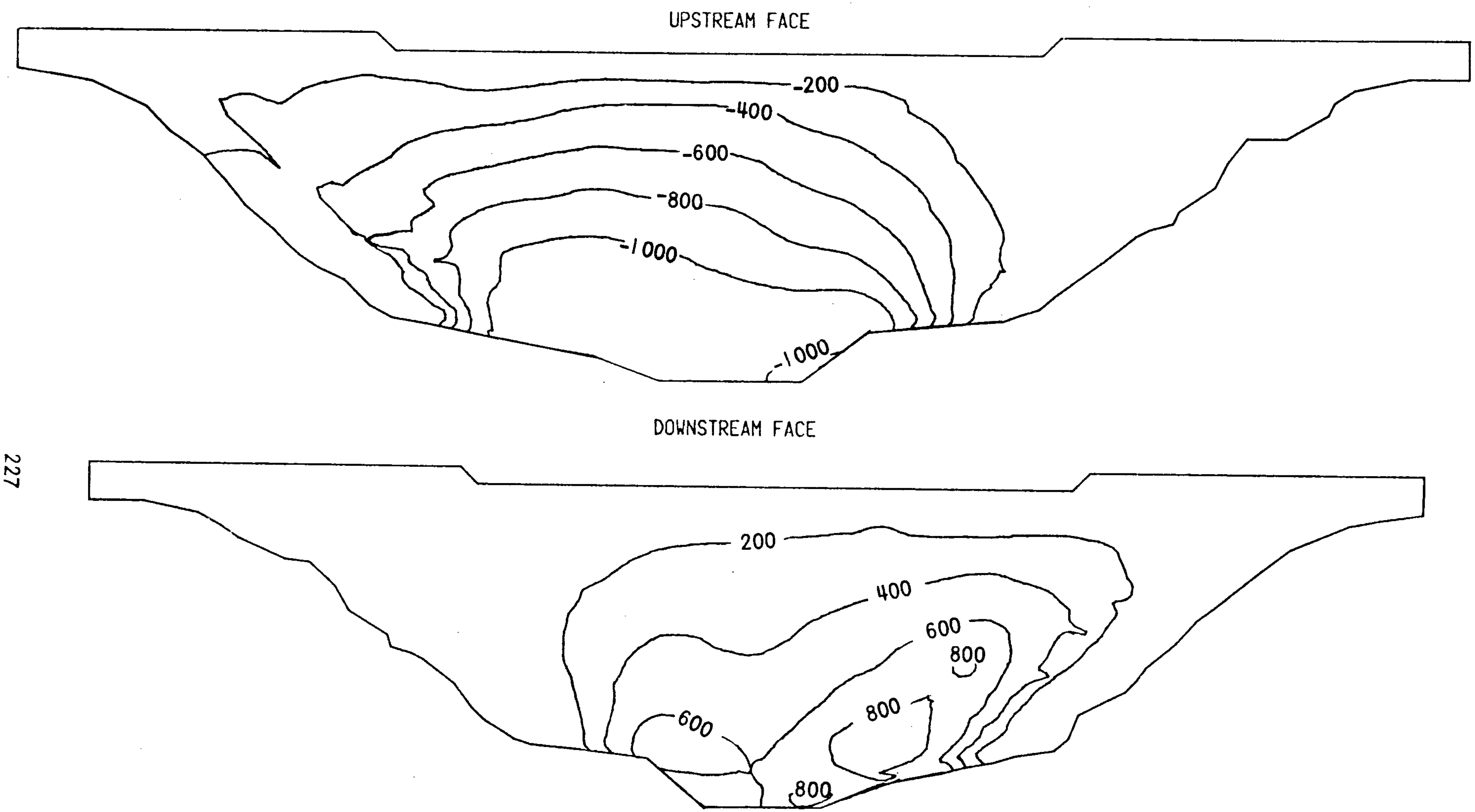


FIGURE 9-50 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.66 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.
NOTE: STRESSES ARE IN P.S.I.

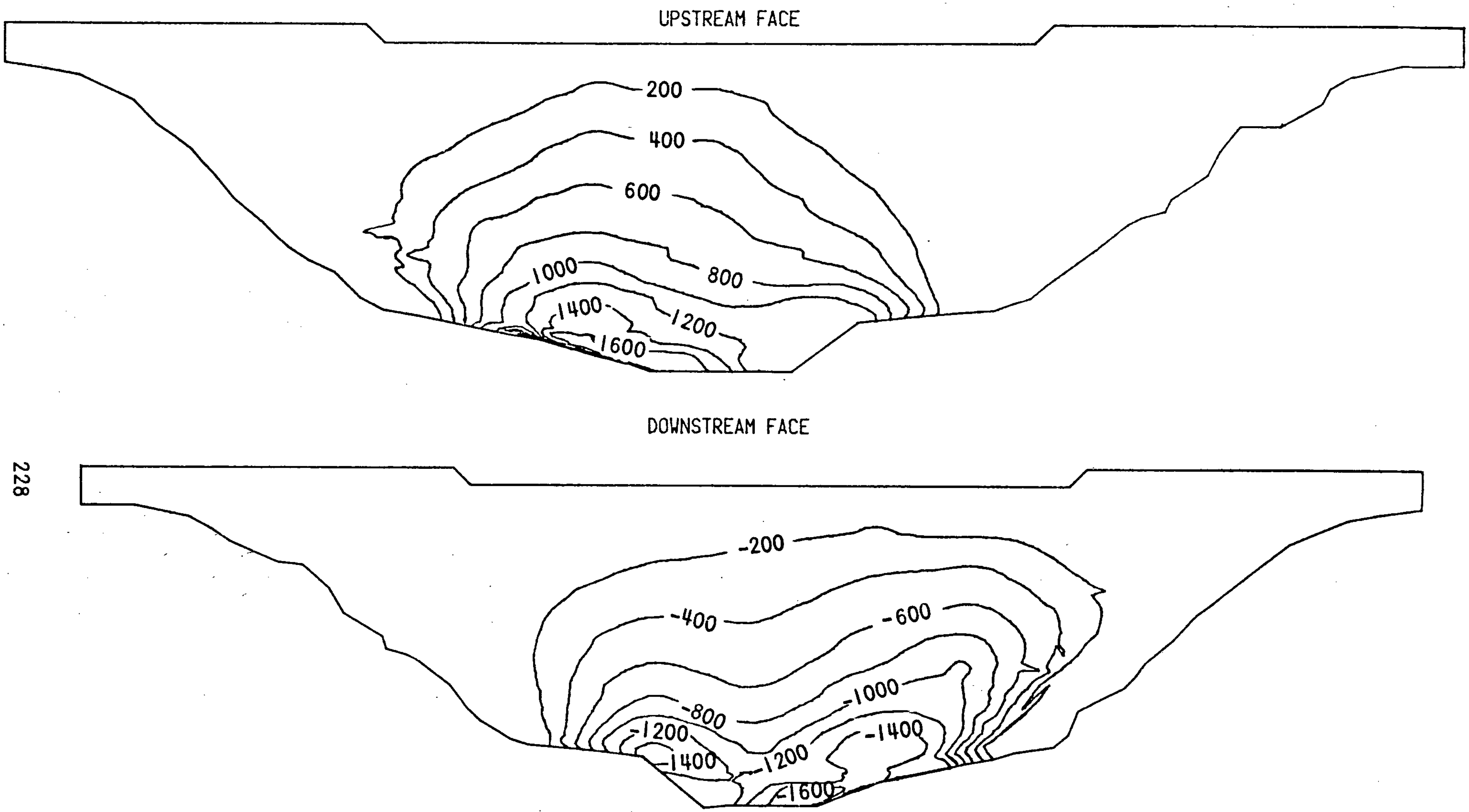


FIGURE 9-51 INSTANTANEOUS VALUES OF CANTILEVER STRESSES (AT 4.78 SEC.) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET) DUE TO QUAKE 2. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. WAVE REFLECTION COEFFICIENT = 0.90.

NOTE: STRESSES ARE IN P.S.I.

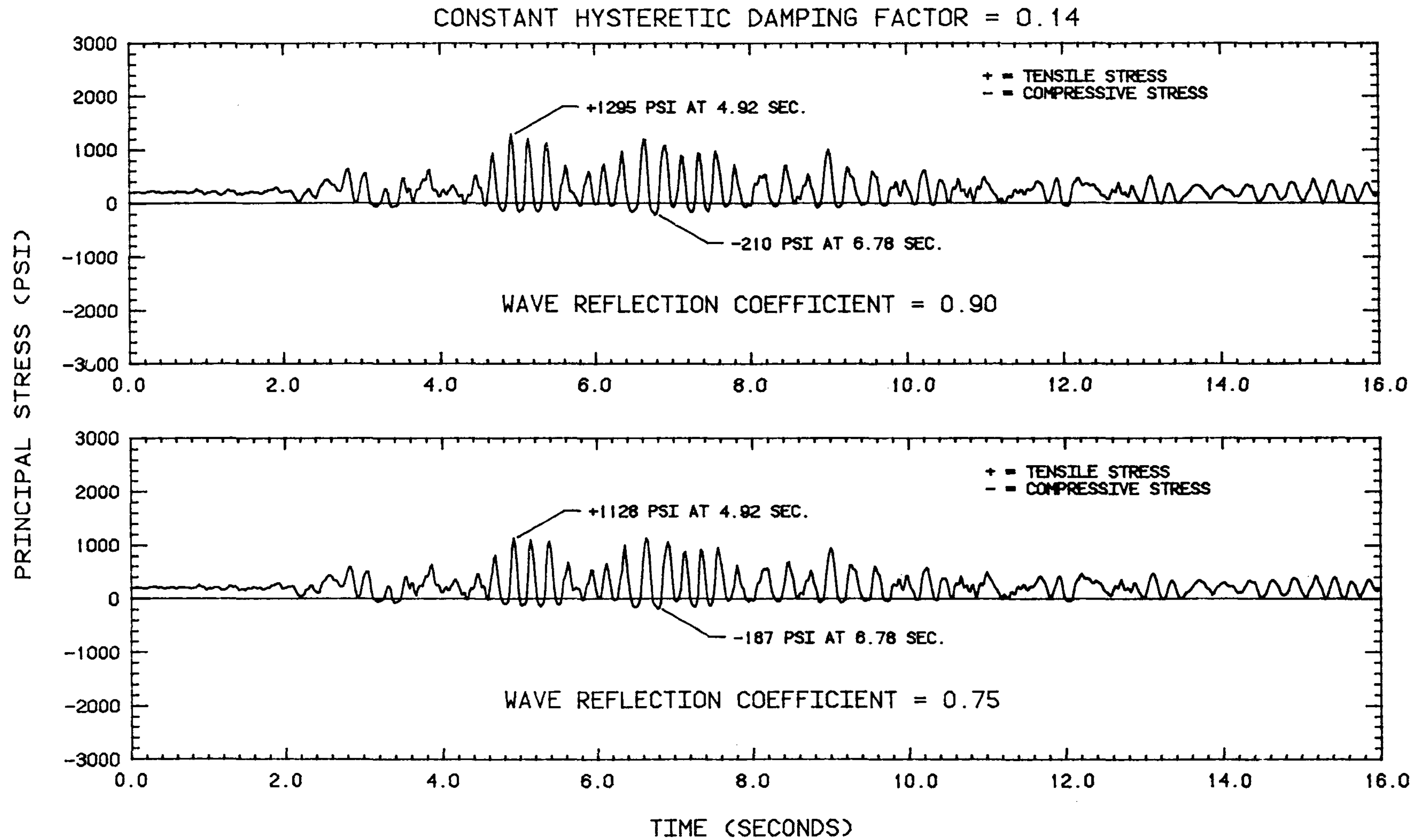


FIGURE 9-52 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 472 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

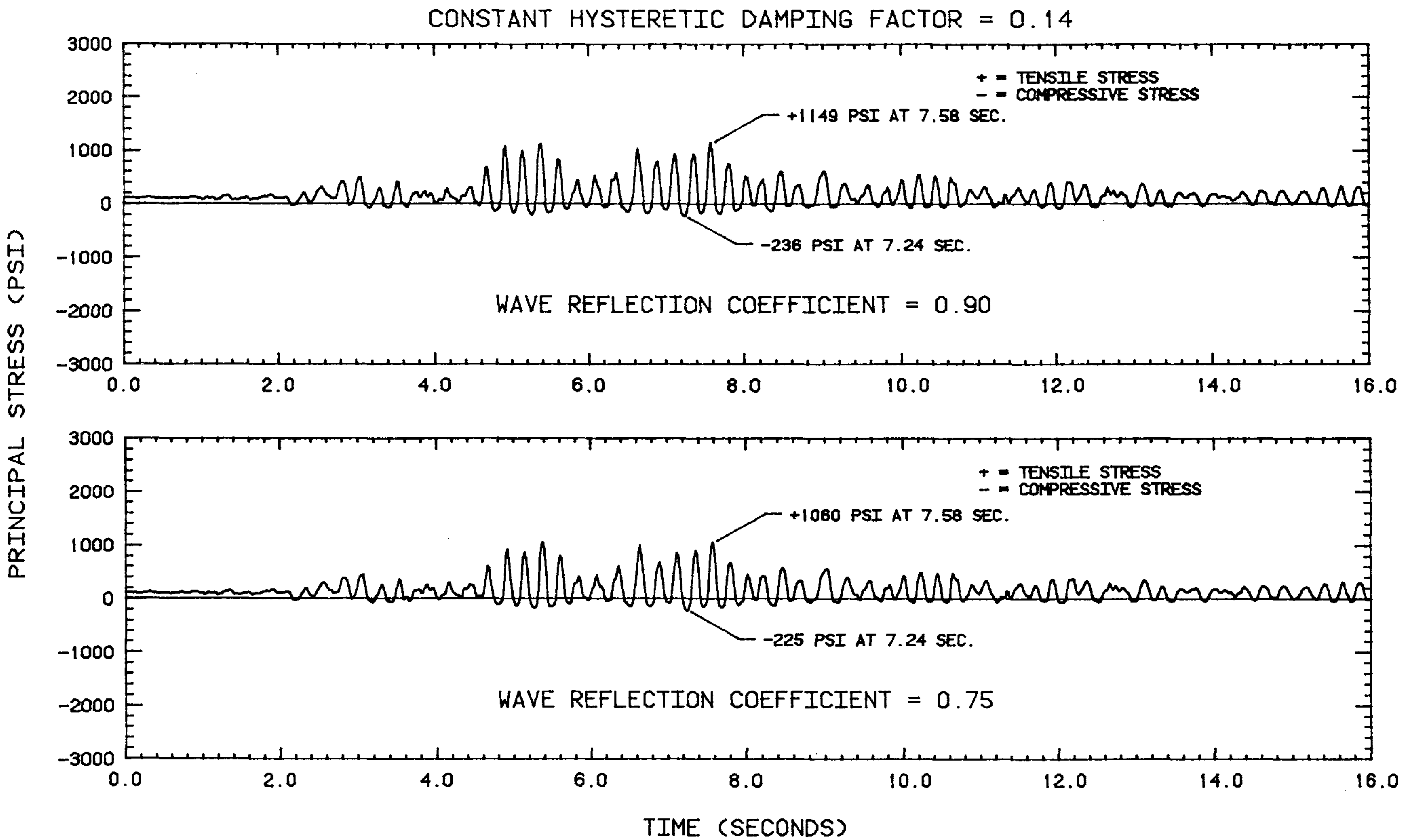


FIGURE 9-53 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 481 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

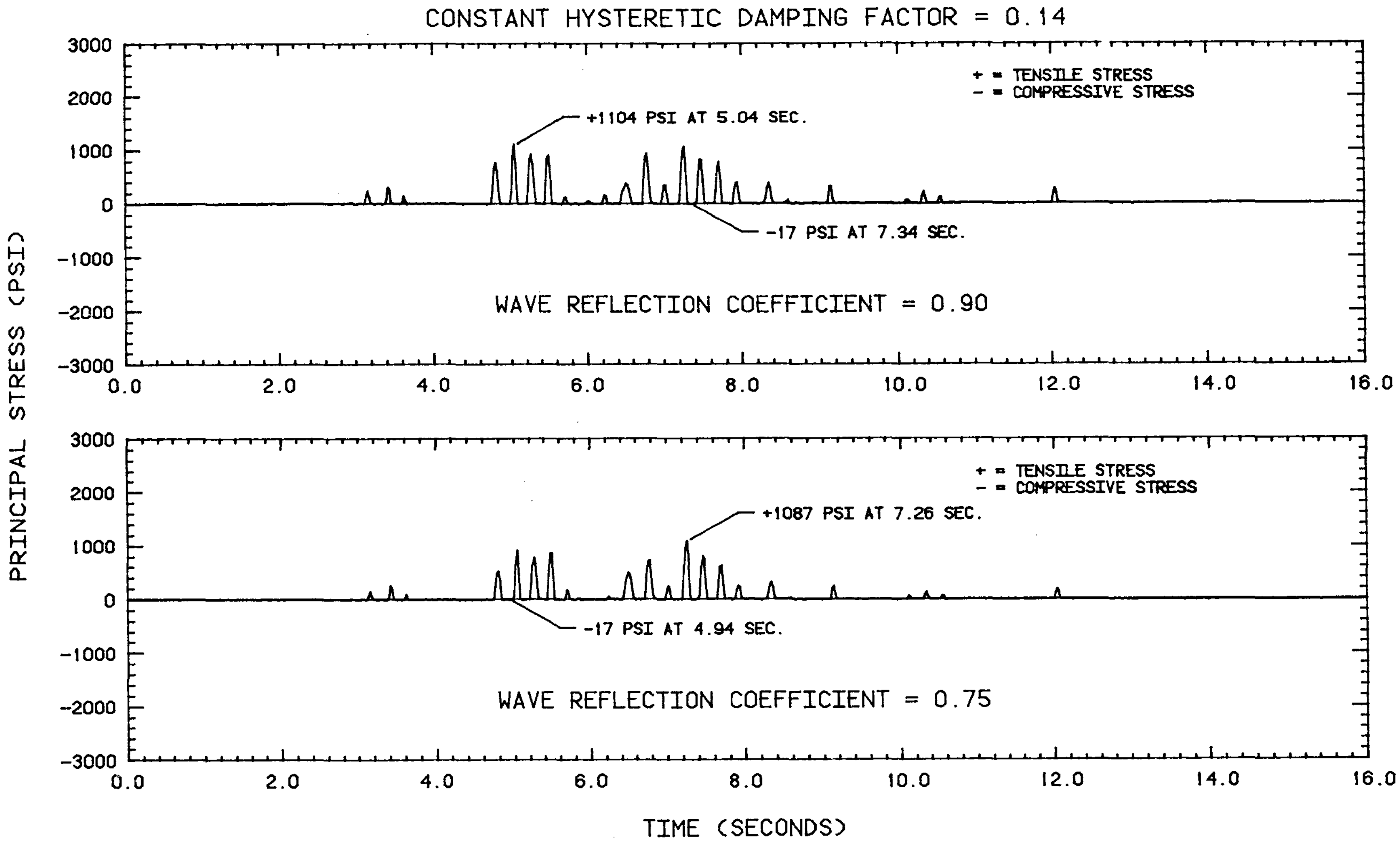


FIGURE 9-54 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 303 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

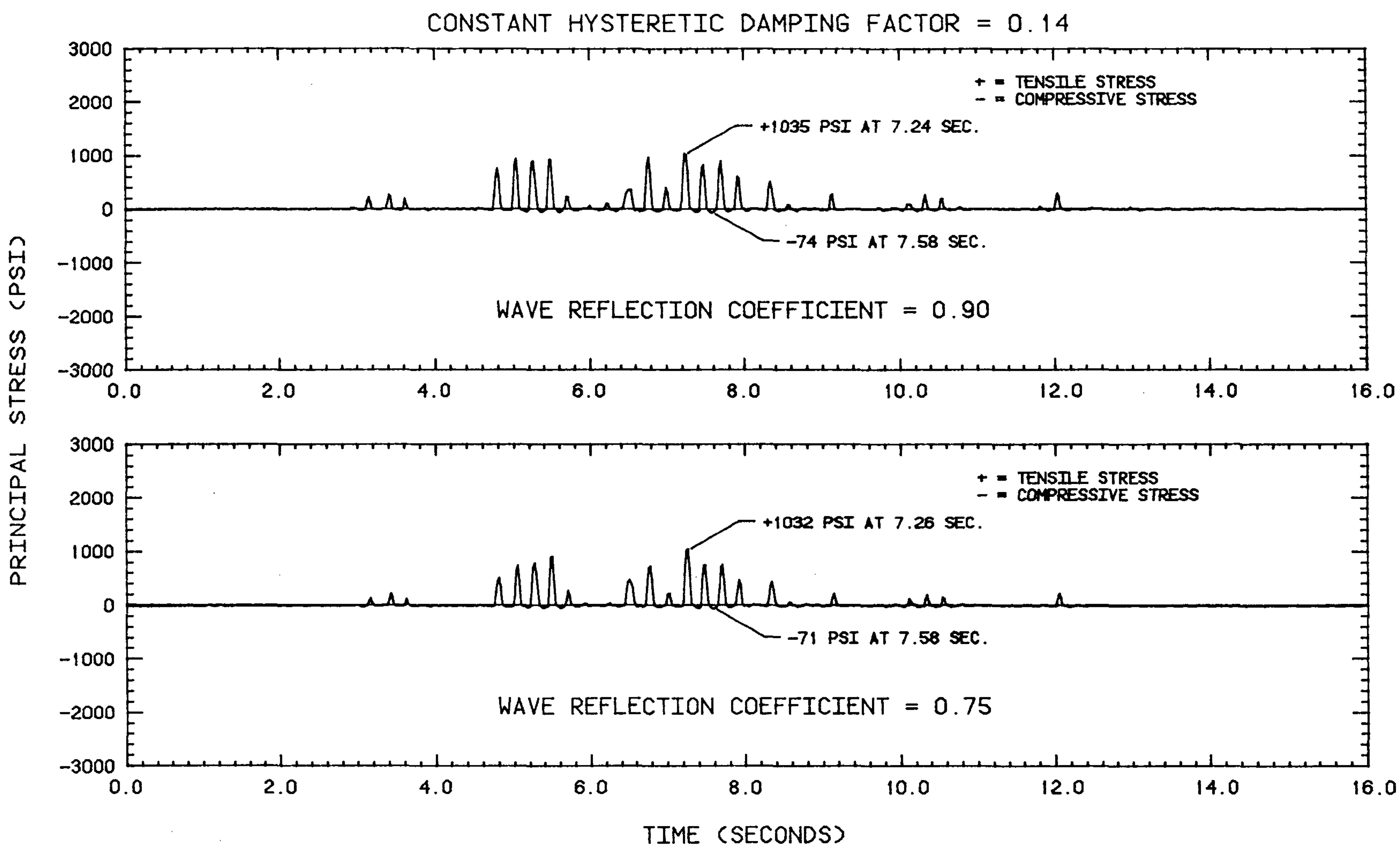


FIGURE 9-55 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 304 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

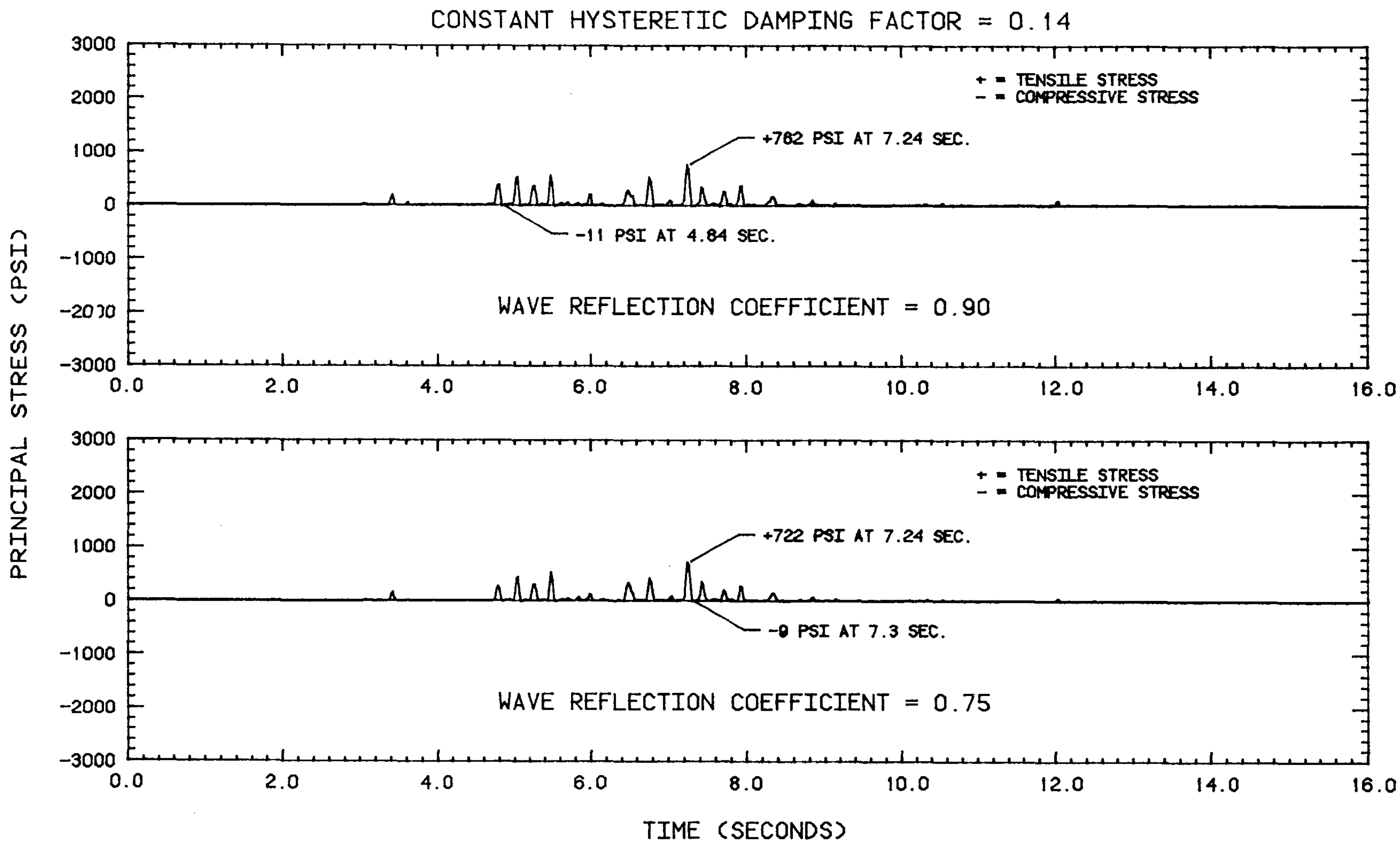


FIGURE 9-56 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 22 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

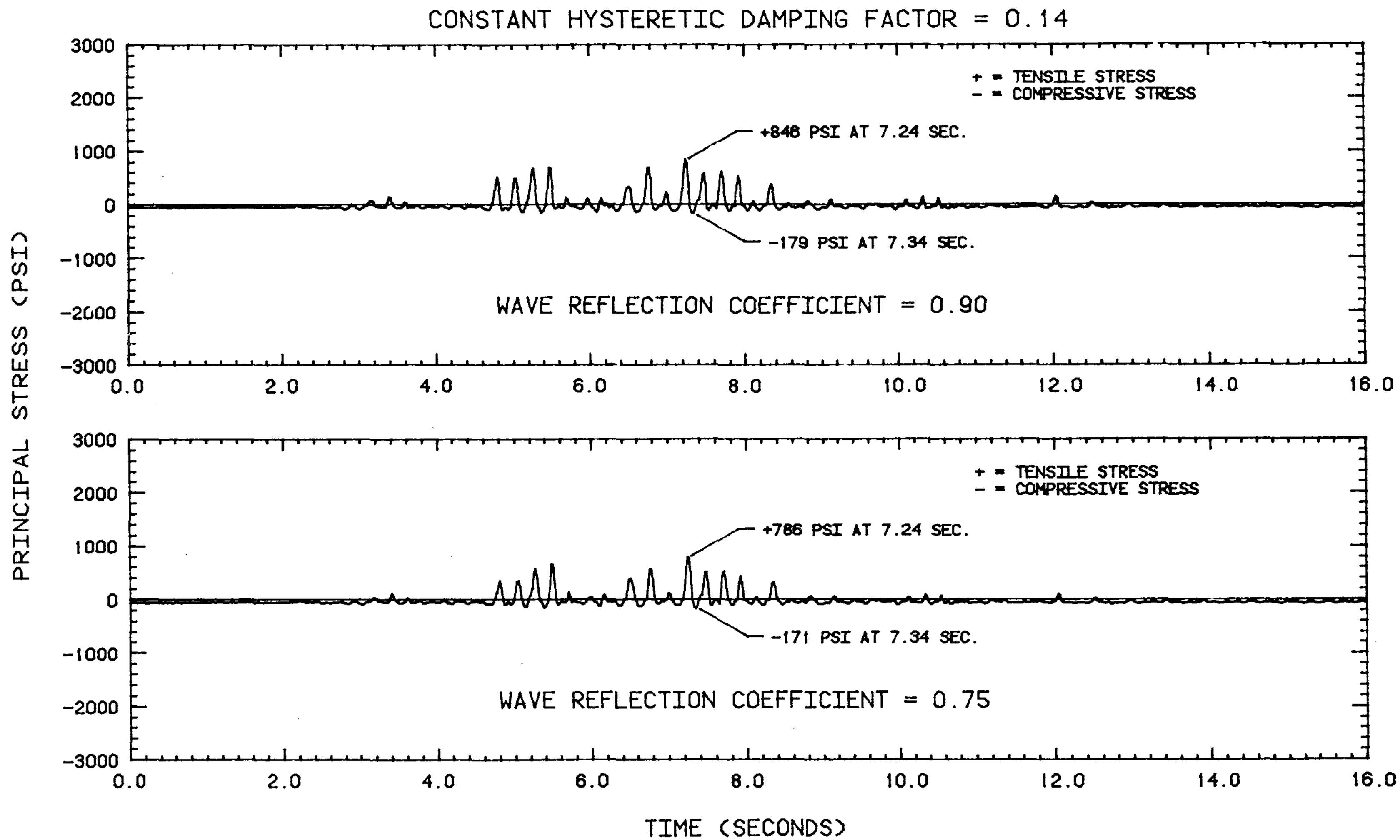


FIGURE 9-57 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

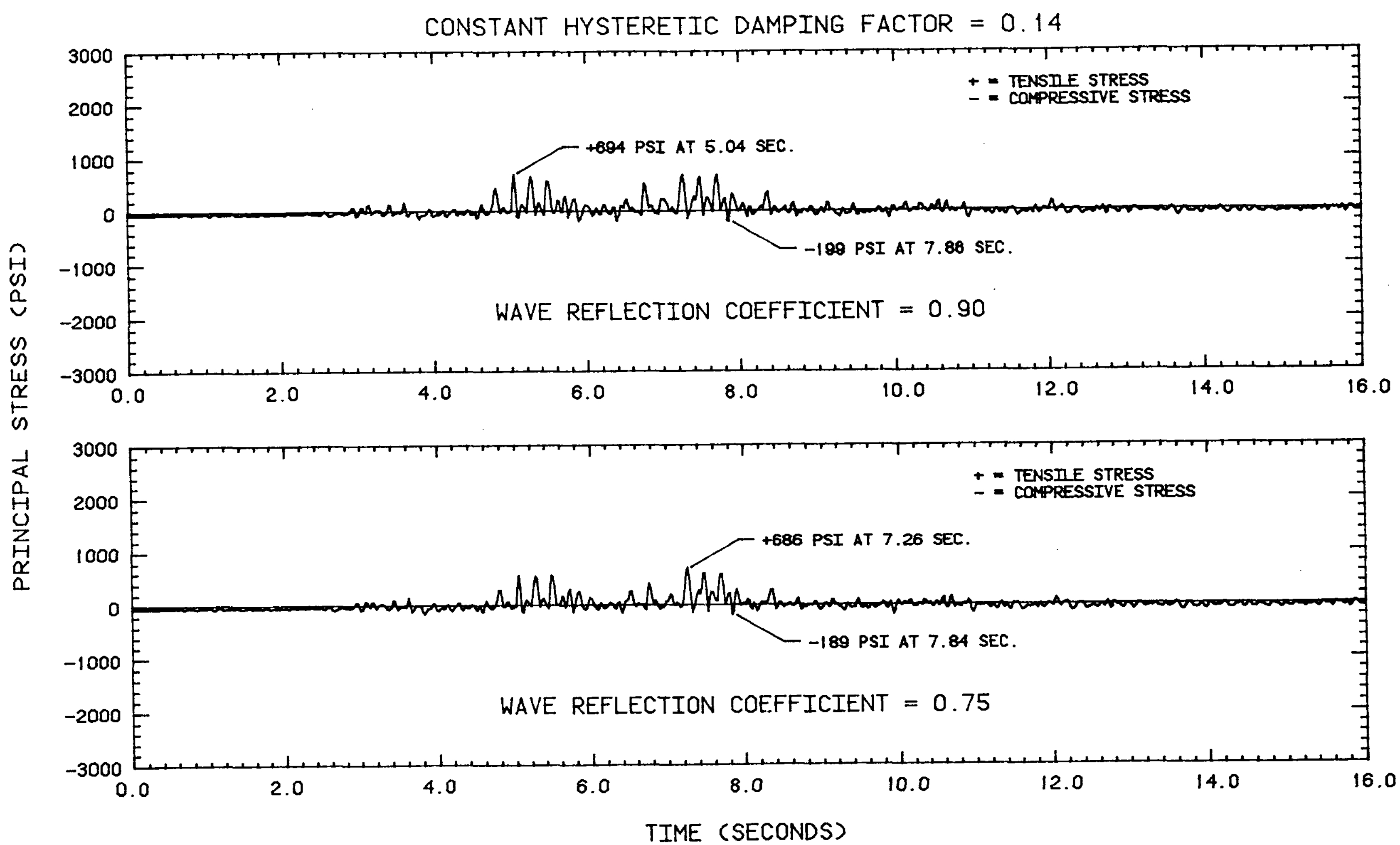


FIGURE 9-58 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 114 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

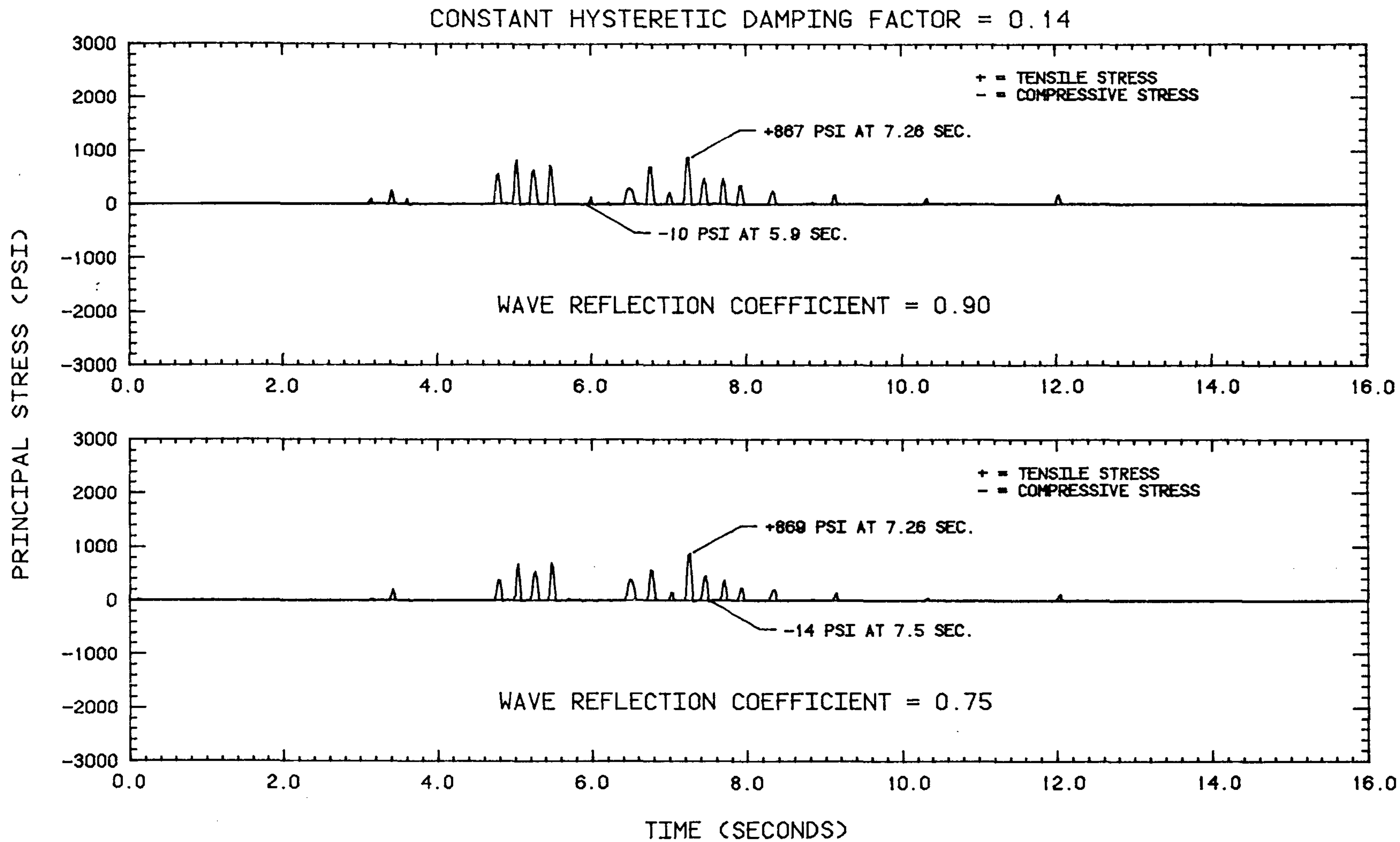


FIGURE 9-59 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 217 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

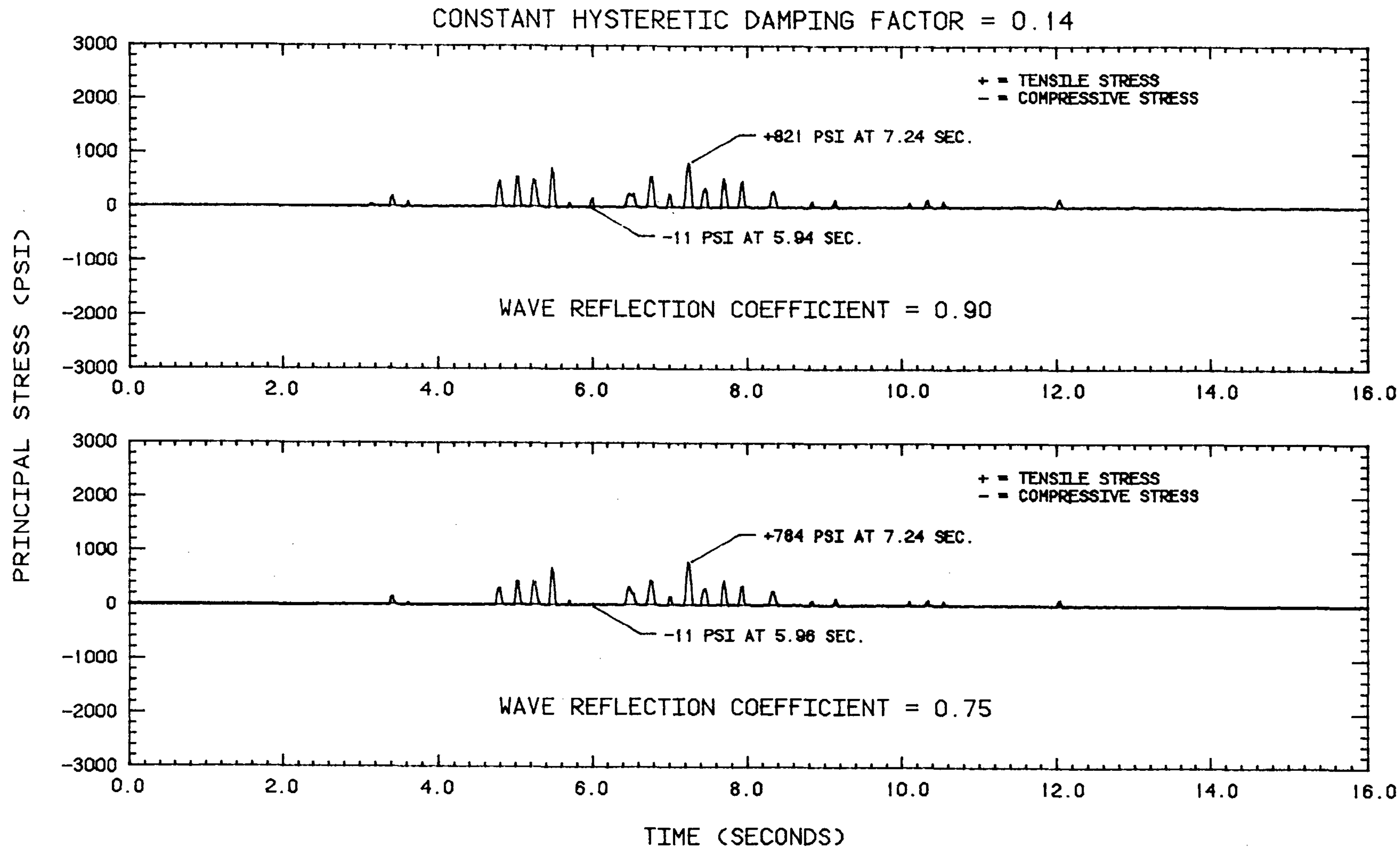


FIGURE 9-60 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 218 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

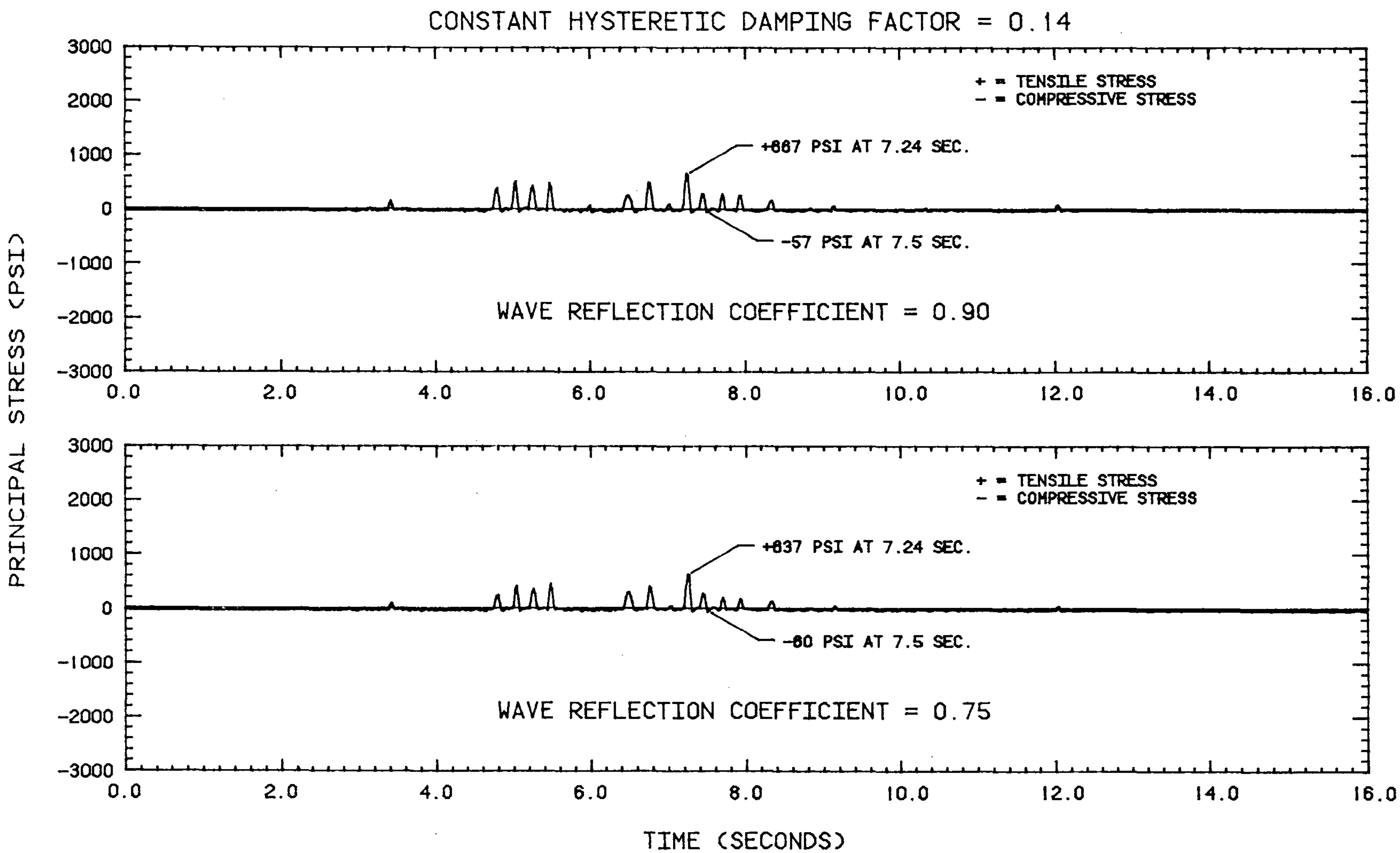


FIGURE 9-61 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 227 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

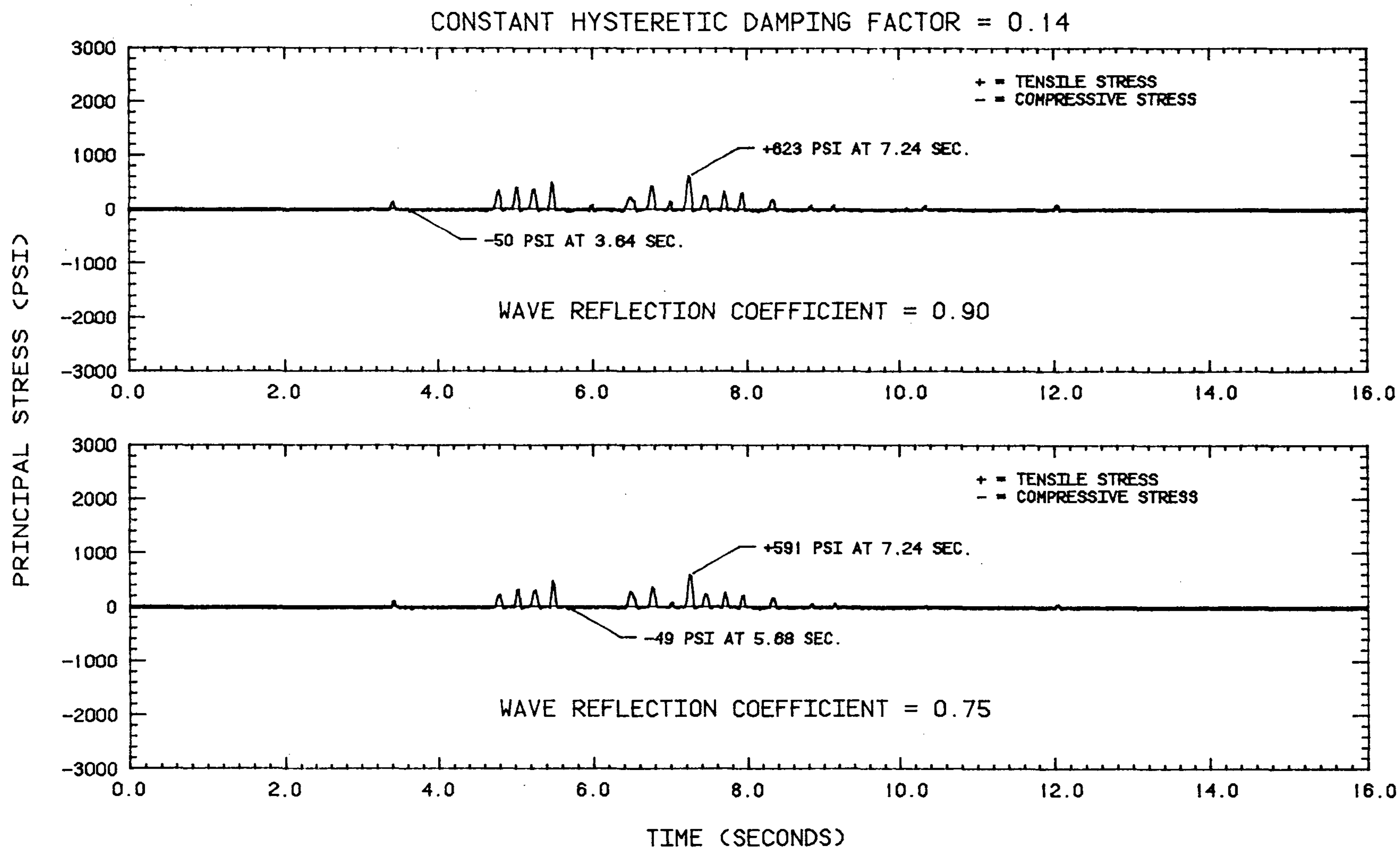


FIGURE 9-62 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 228 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

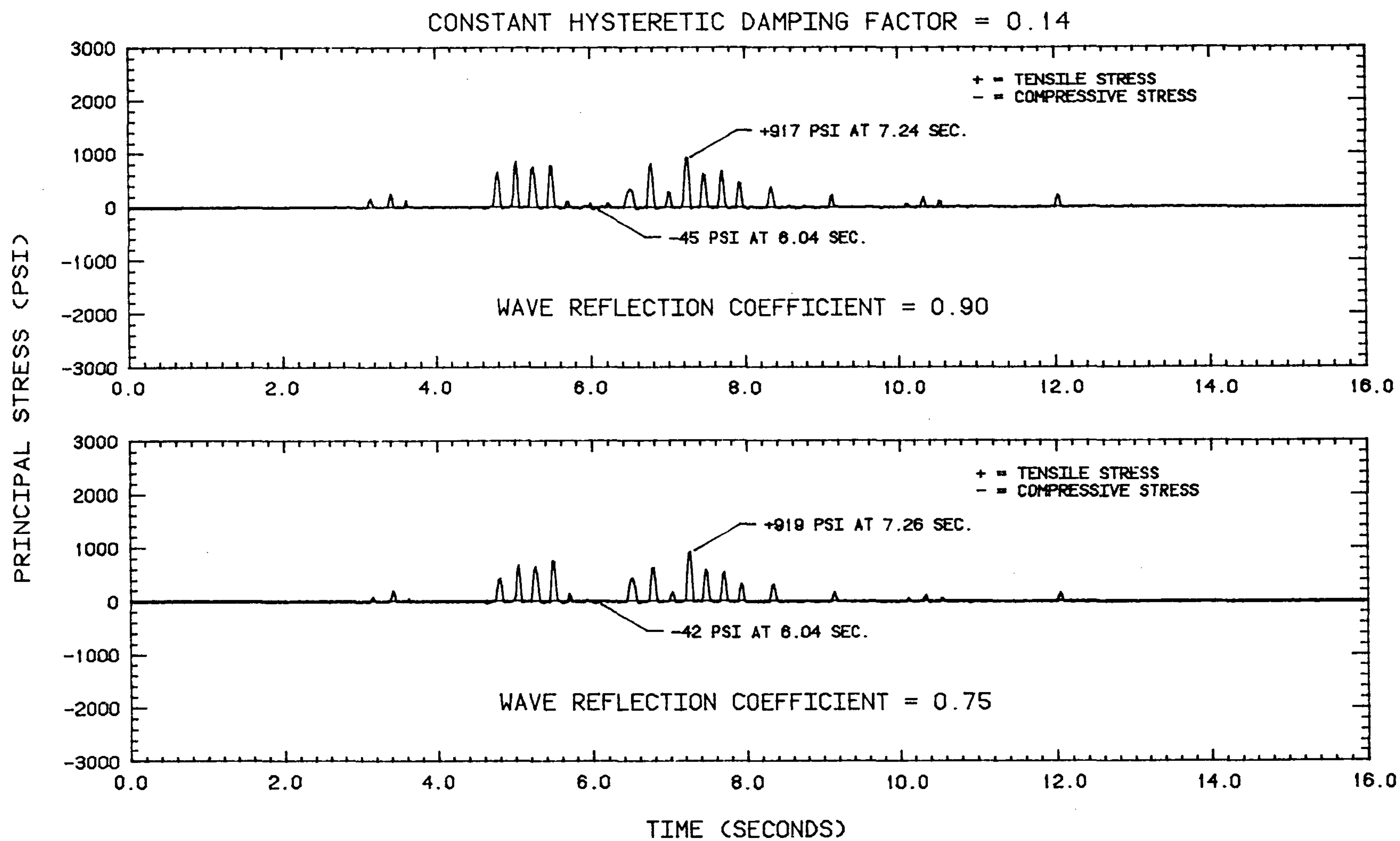


FIGURE 9-63 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

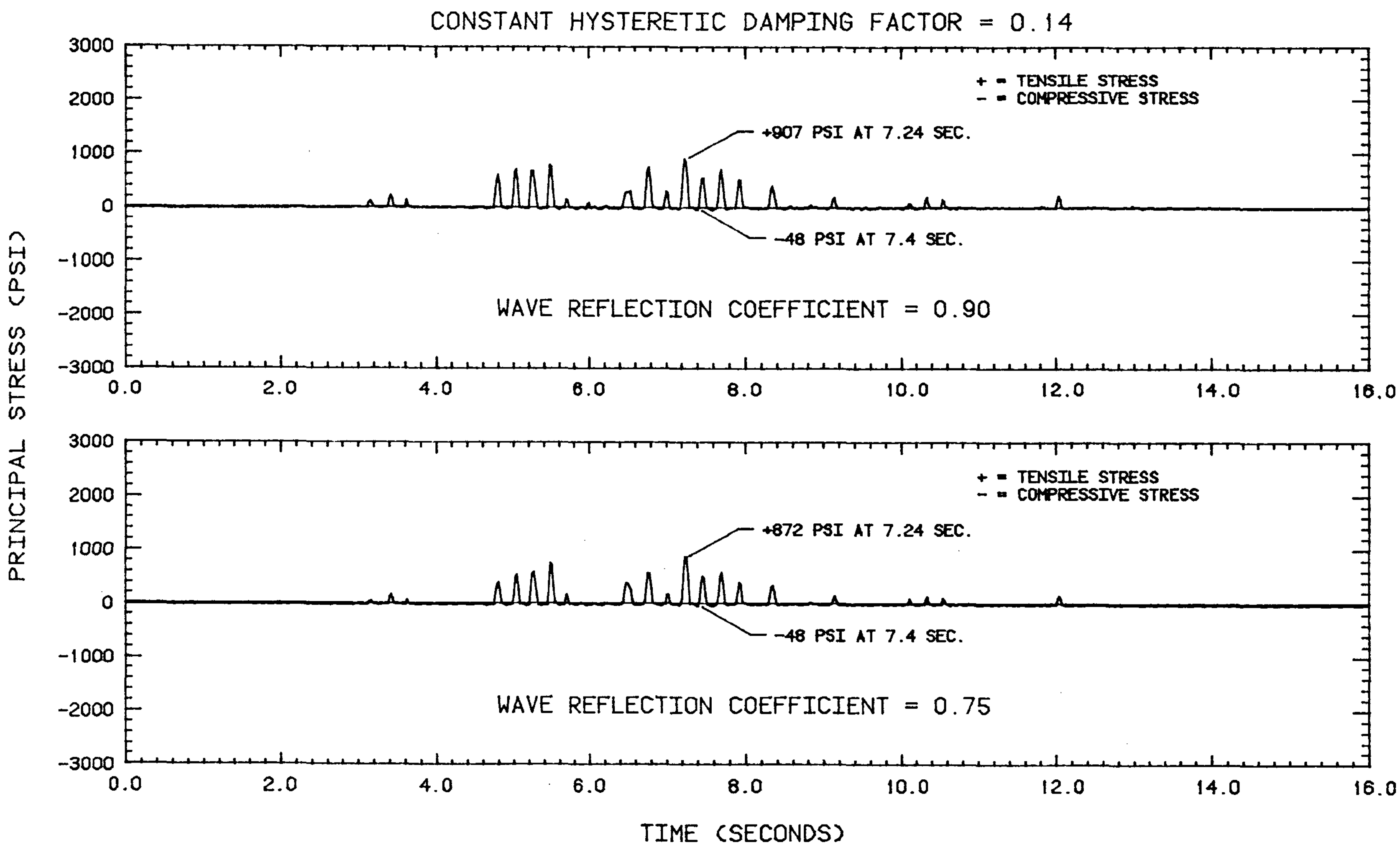


FIGURE 9-64 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 1 IN ELEMENT 9 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

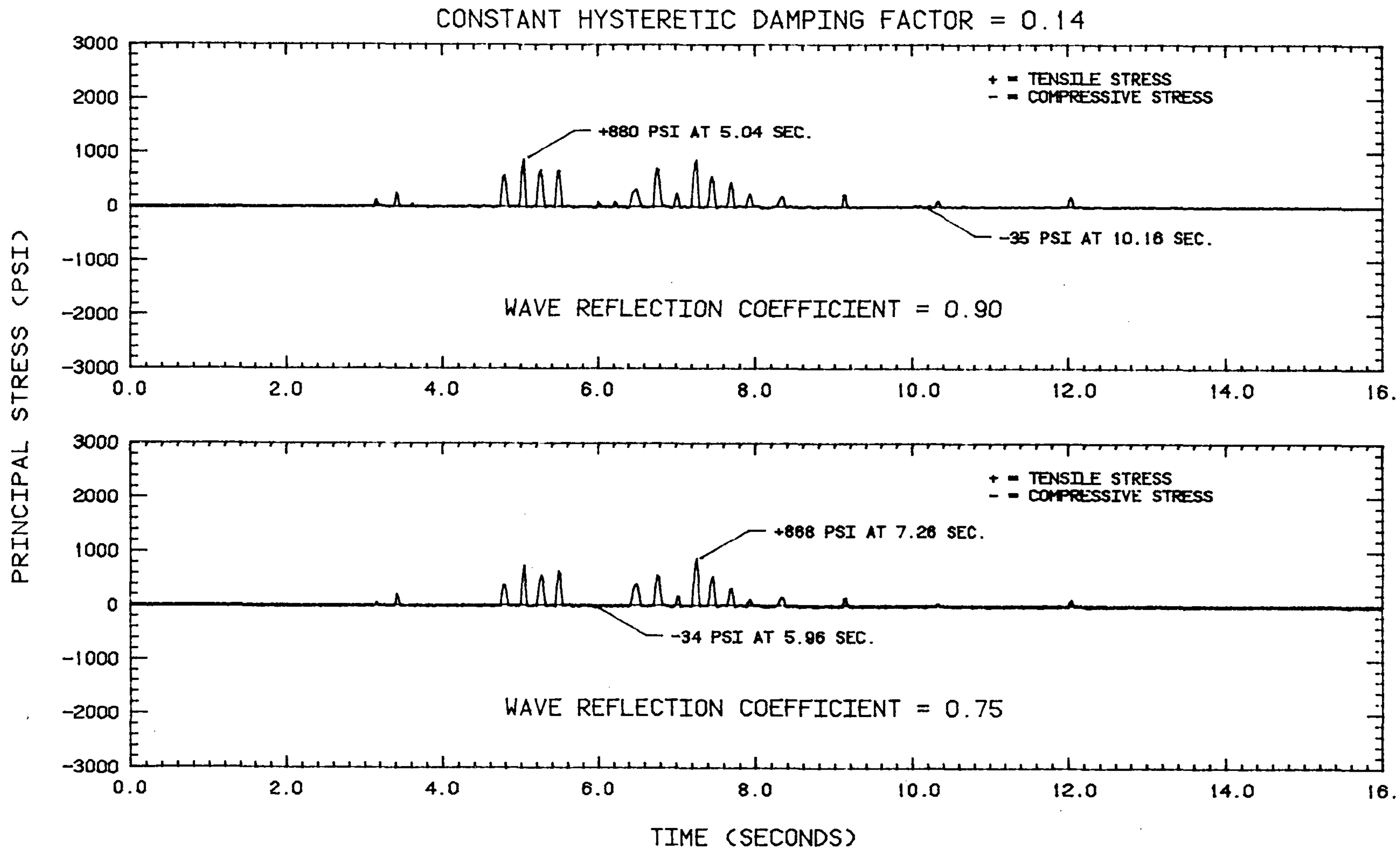


FIGURE 9-65 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 1 IN ELEMENT 8 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

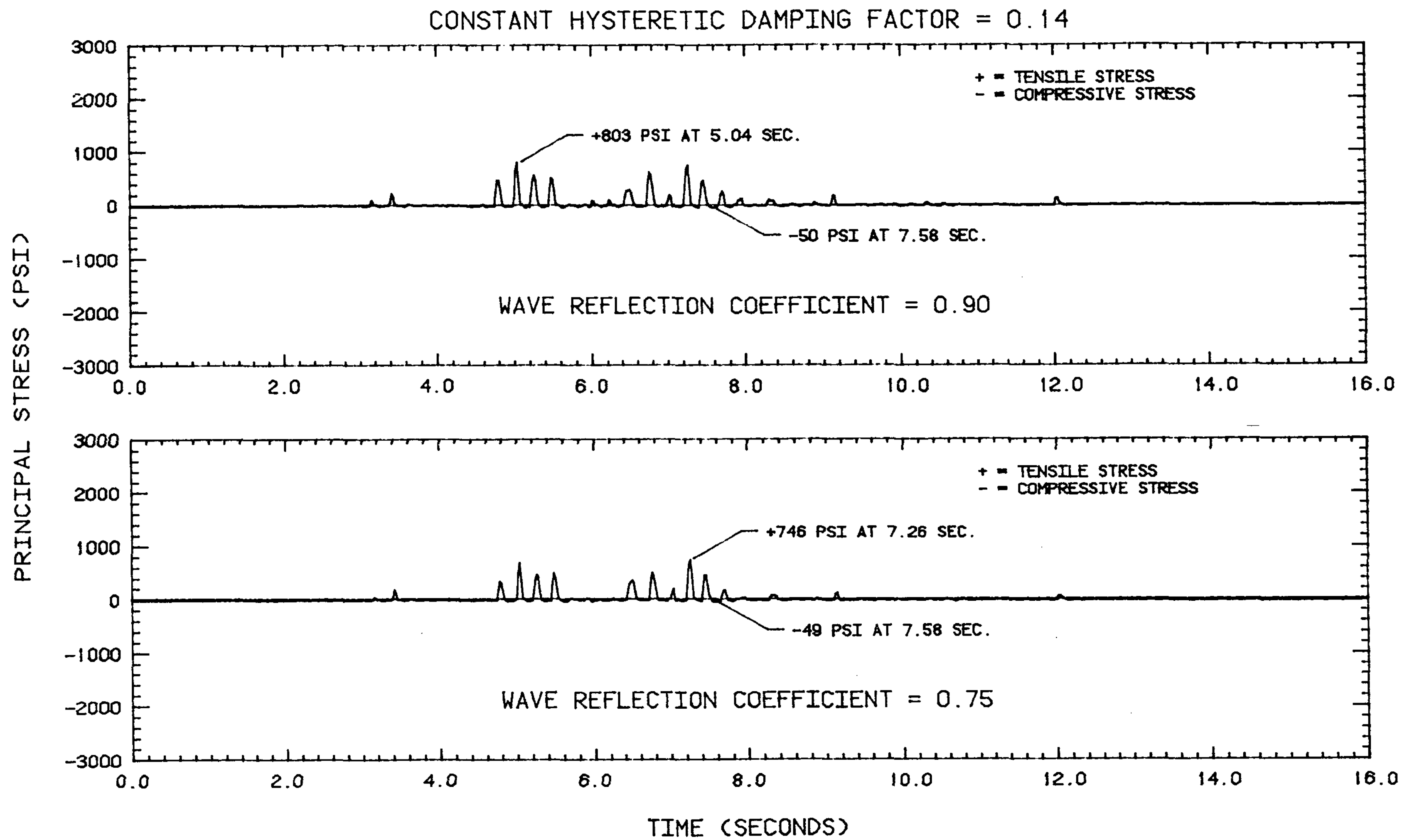


FIGURE 9-66 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 8 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

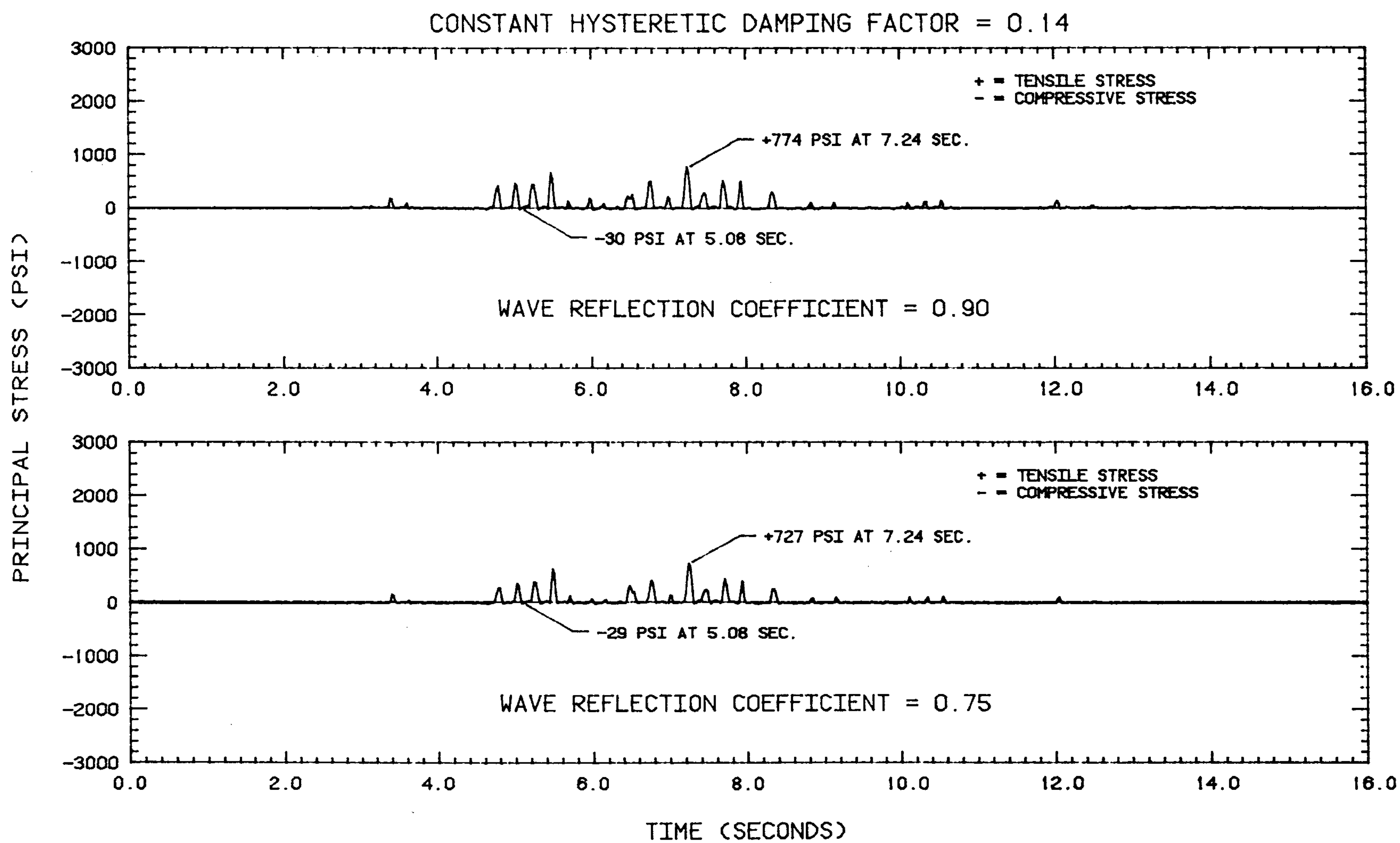


FIGURE 9-67 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 10 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

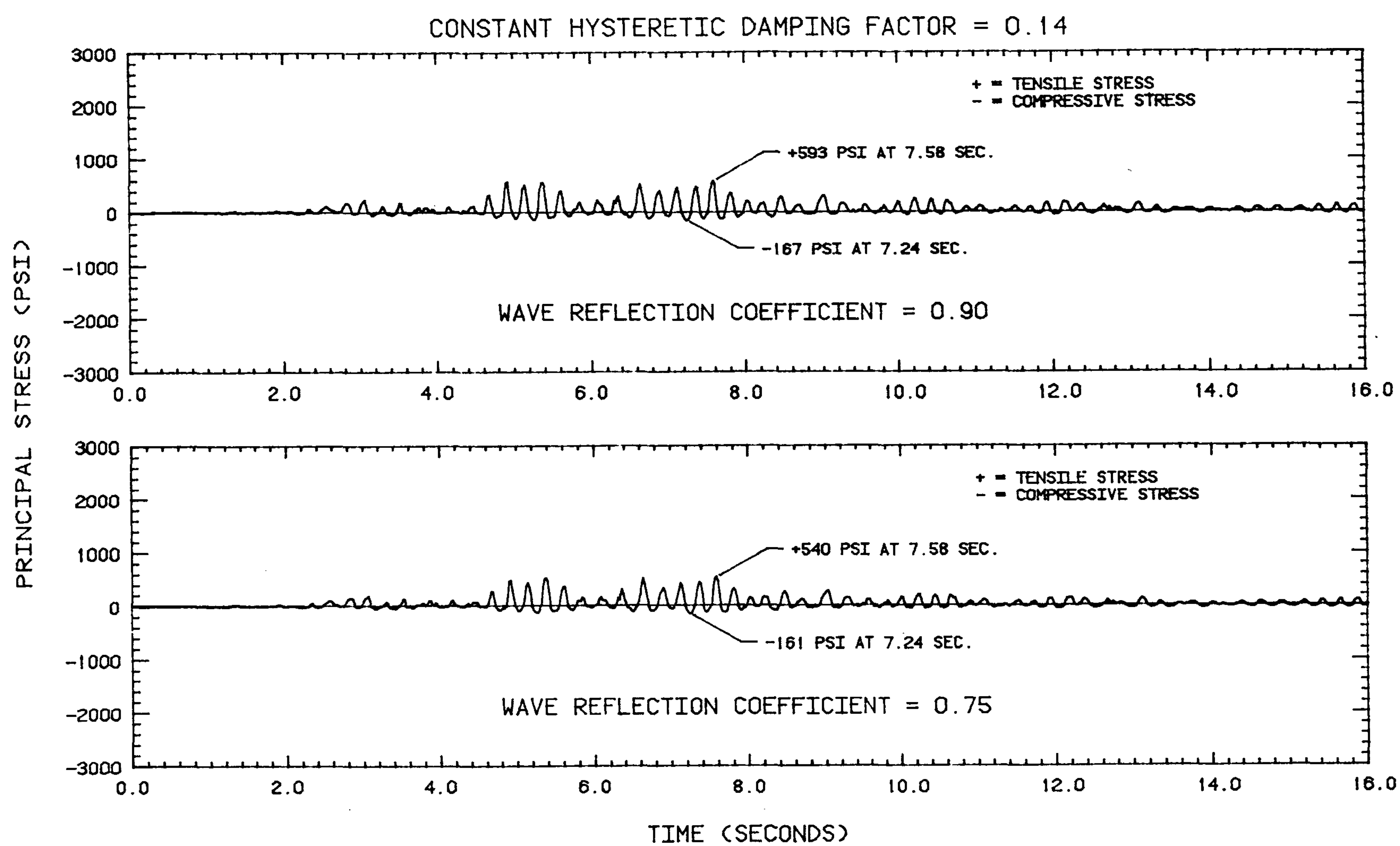


FIGURE 9-68 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 56 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

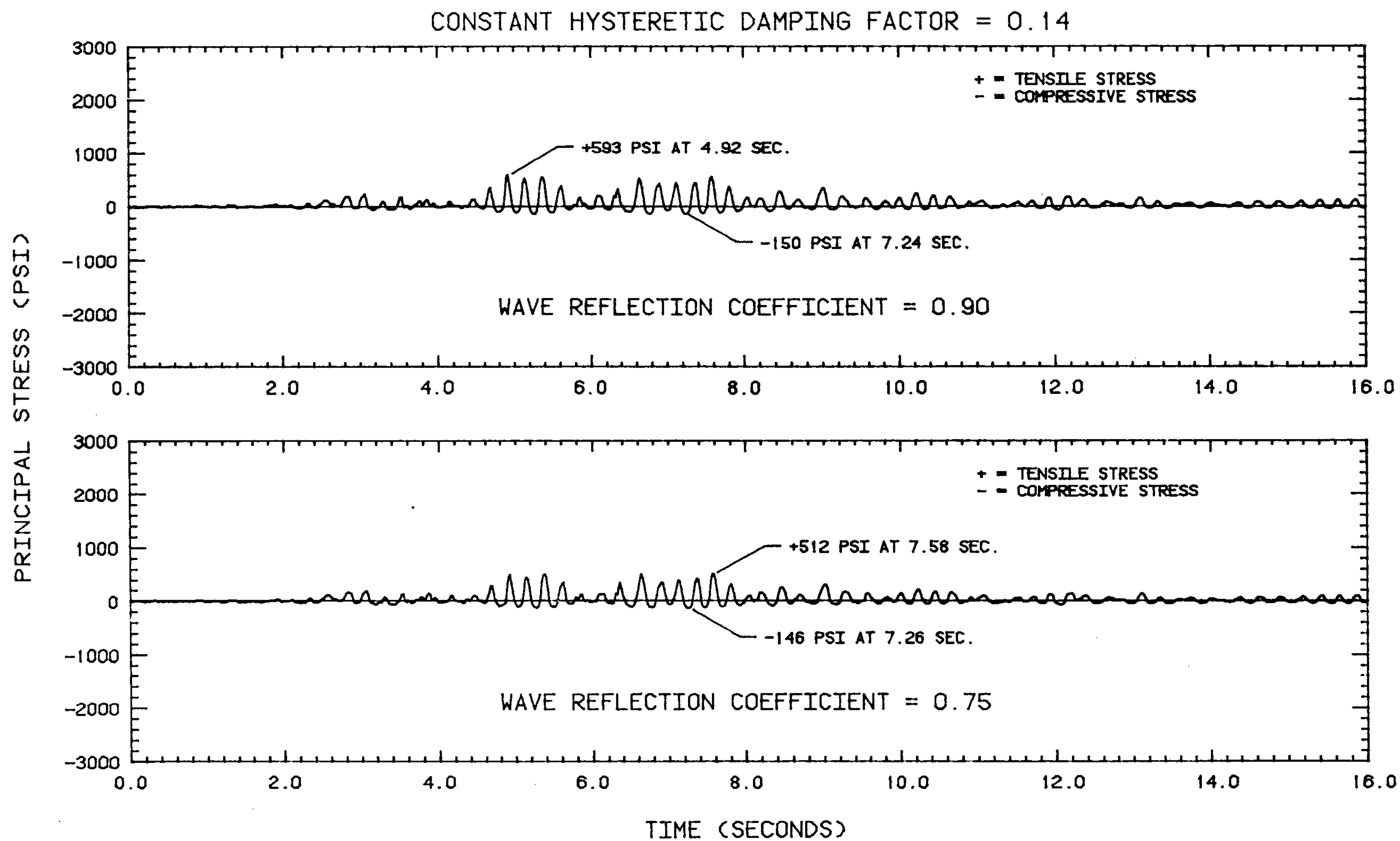


FIGURE 9-69 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 3 IN ELEMENT 56 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

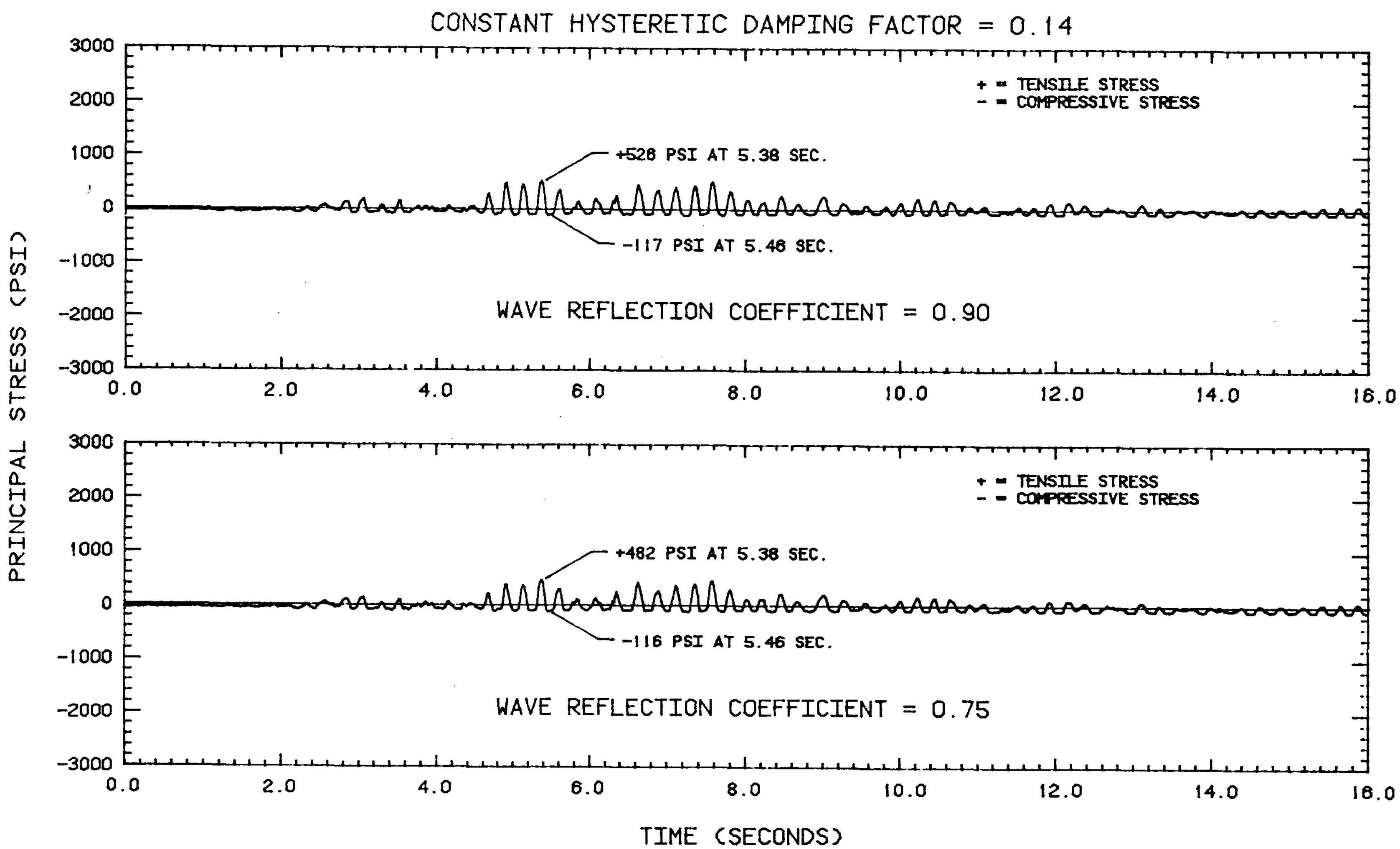


FIGURE 9-70 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 52 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

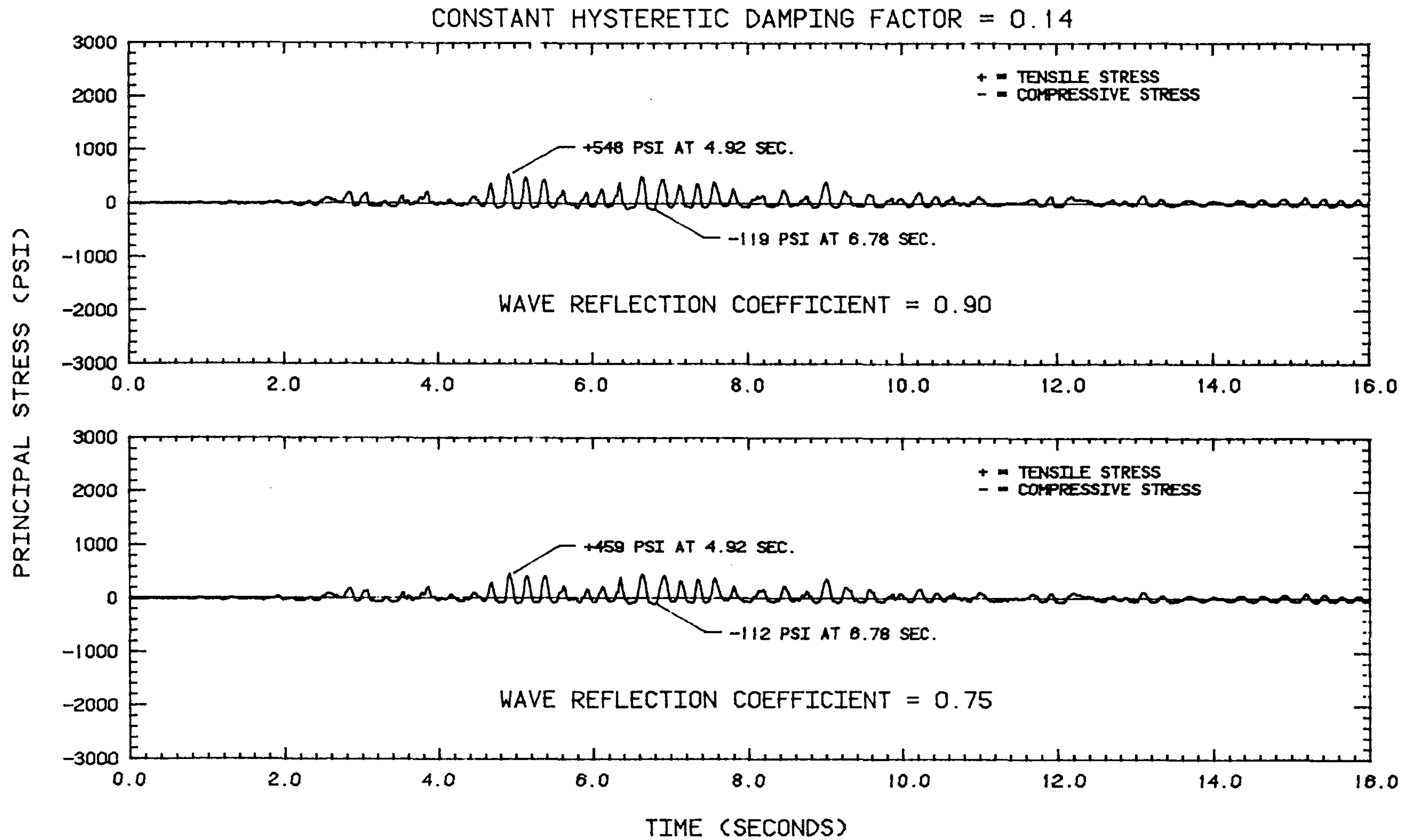


FIGURE 9-71 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 3 IN ELEMENT 51 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

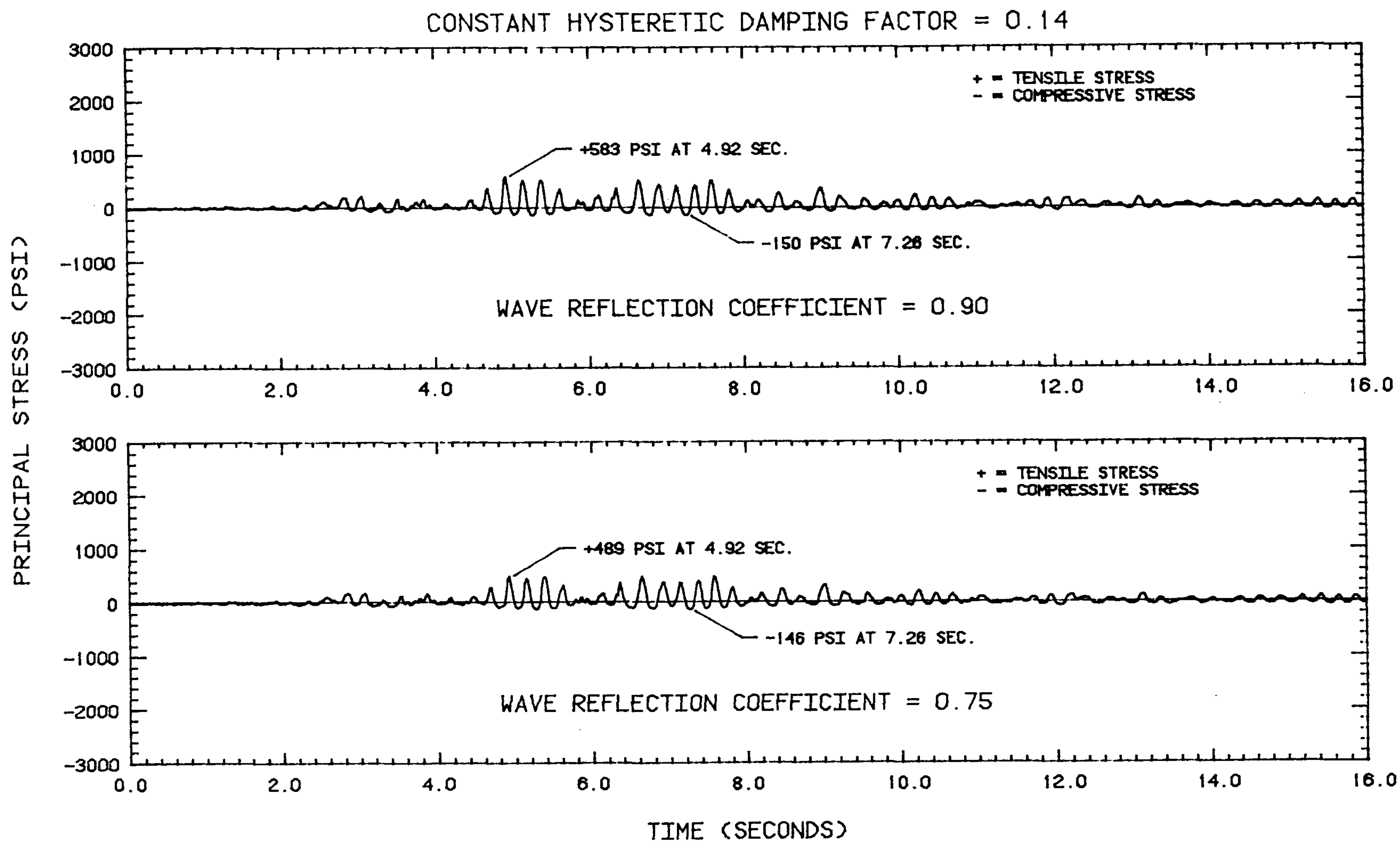


FIGURE 9-72 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 55 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

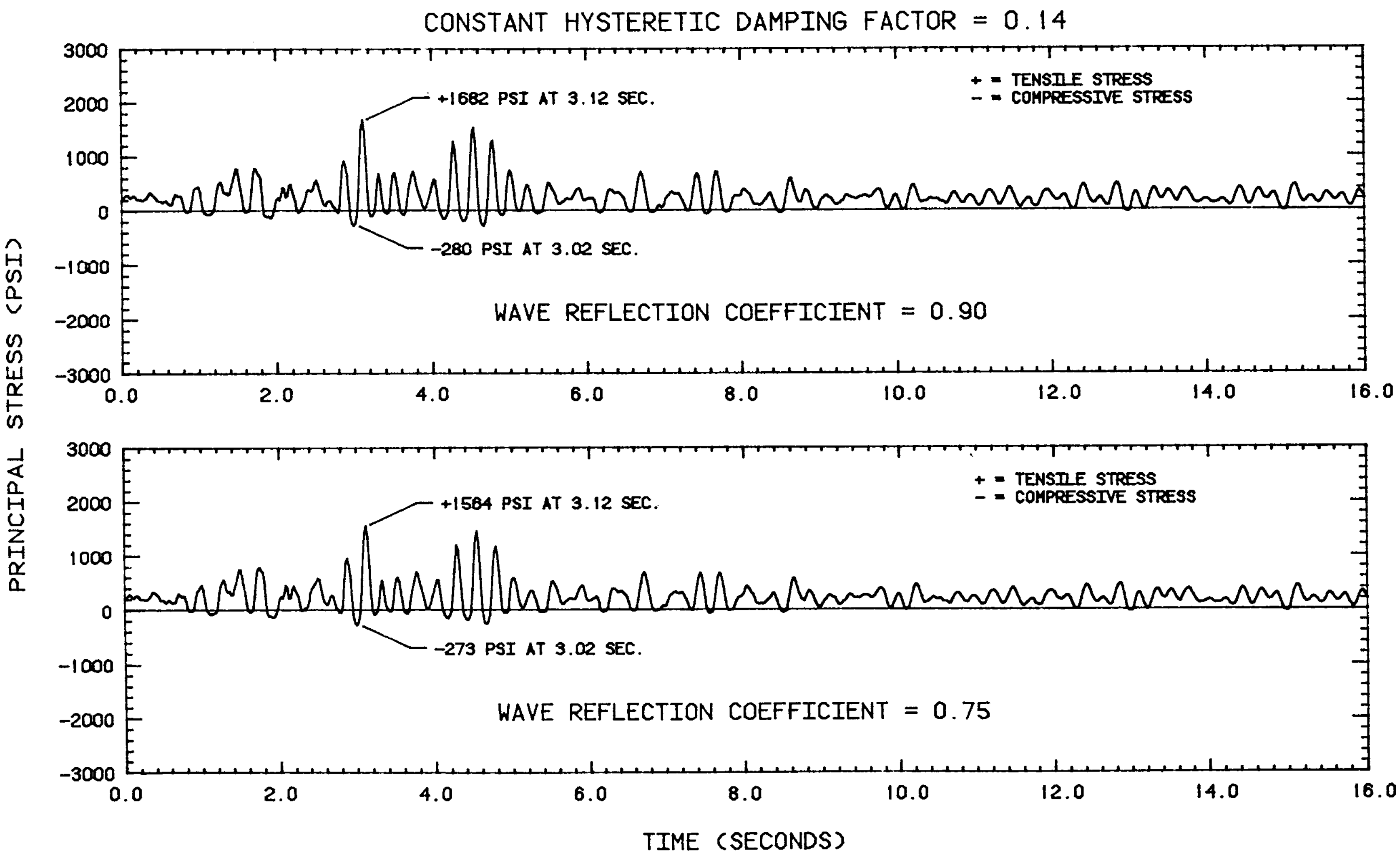


FIGURE 9-73 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 472 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

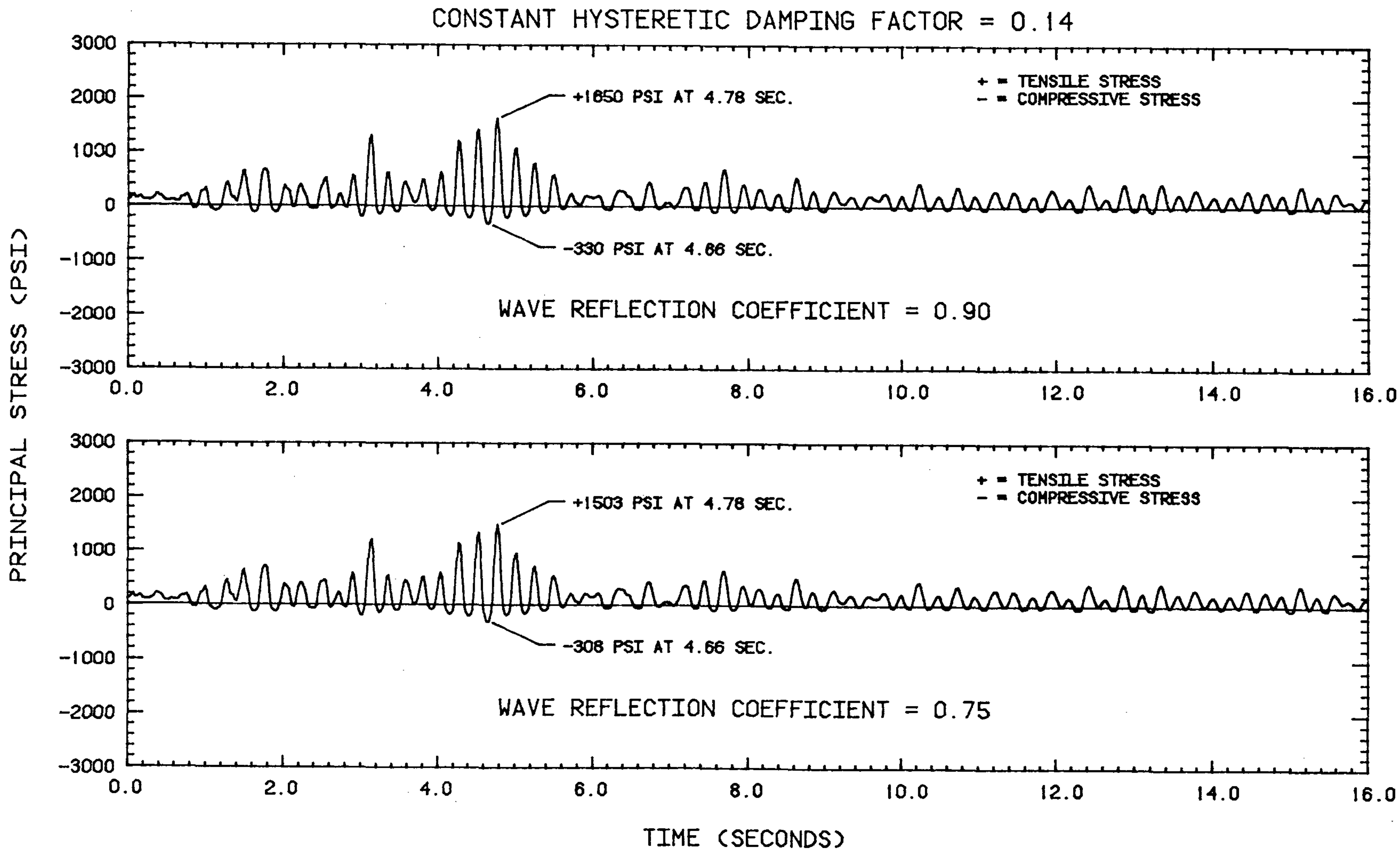


FIGURE 9-74 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 481 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

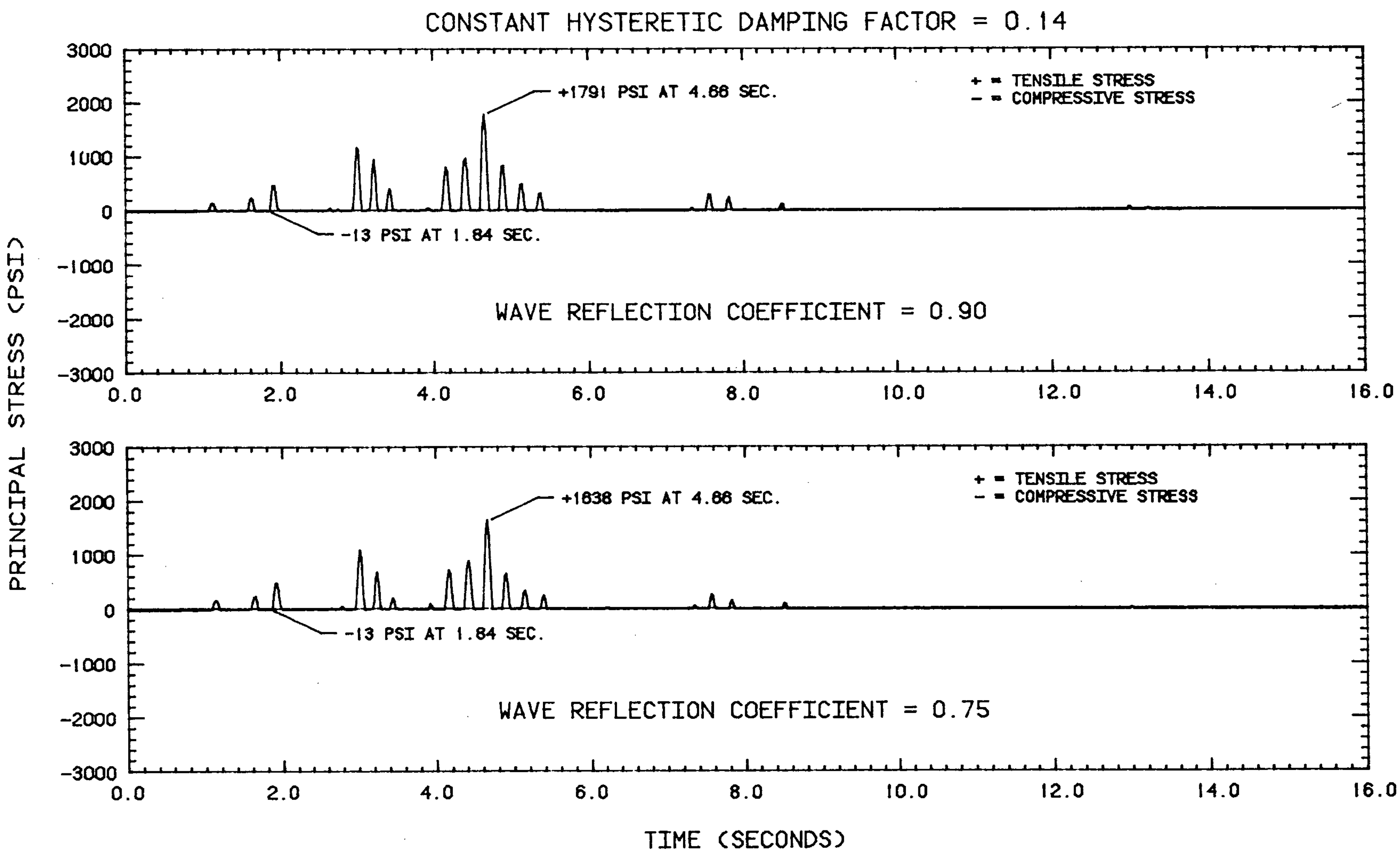


FIGURE 9-75 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 303 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

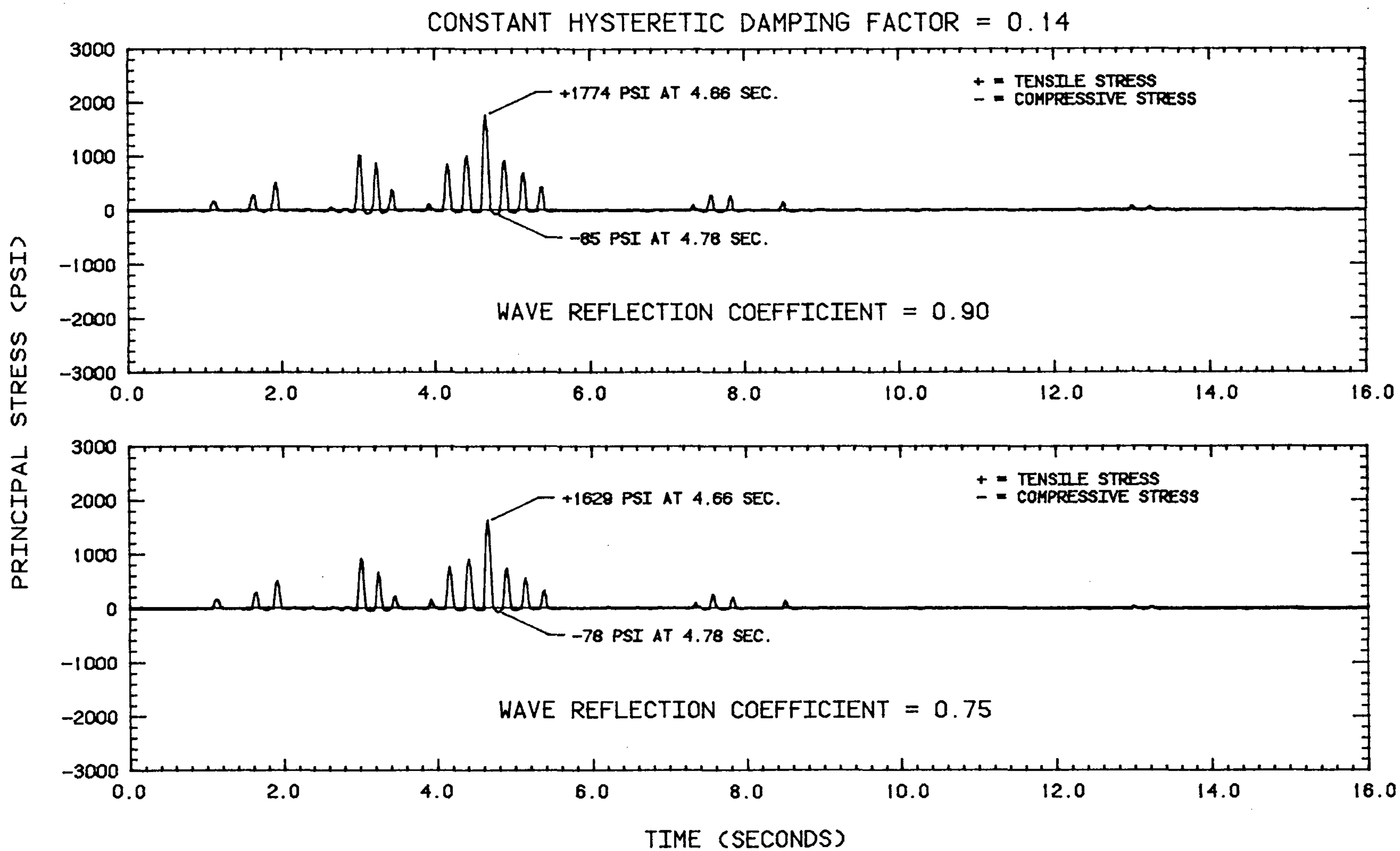


FIGURE 9-76 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 304 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

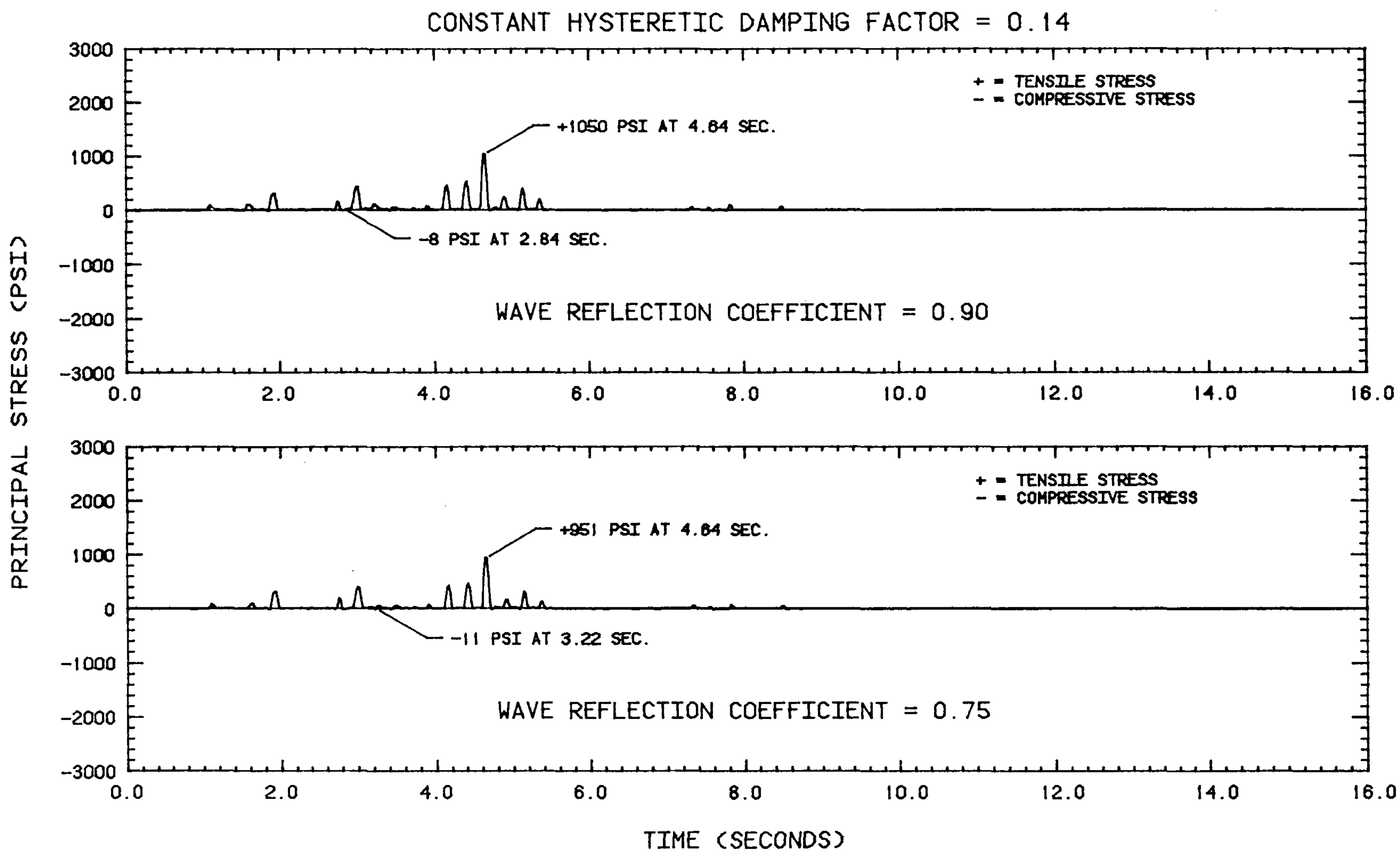


FIGURE 9-77 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 22 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

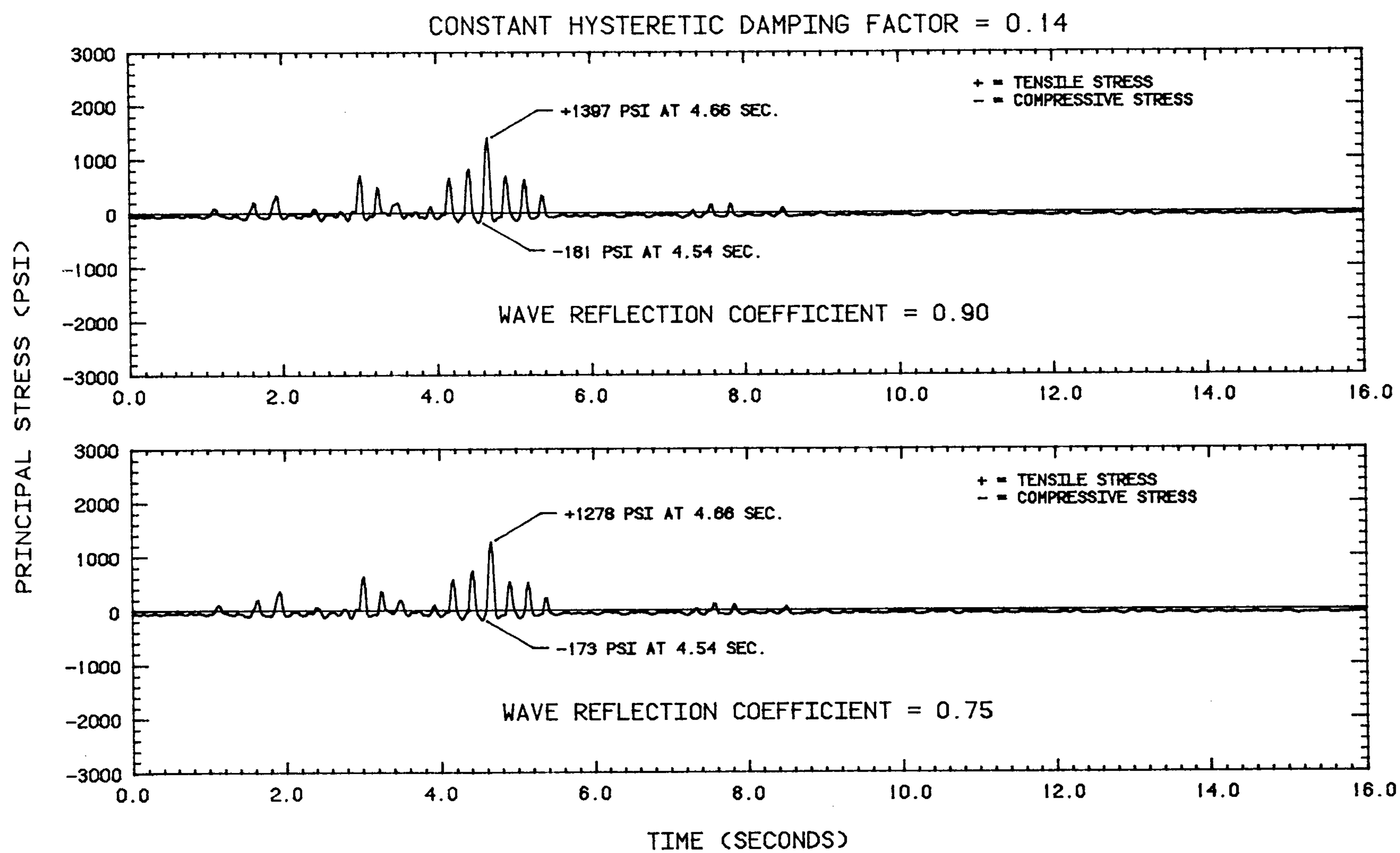


FIGURE 9-78 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

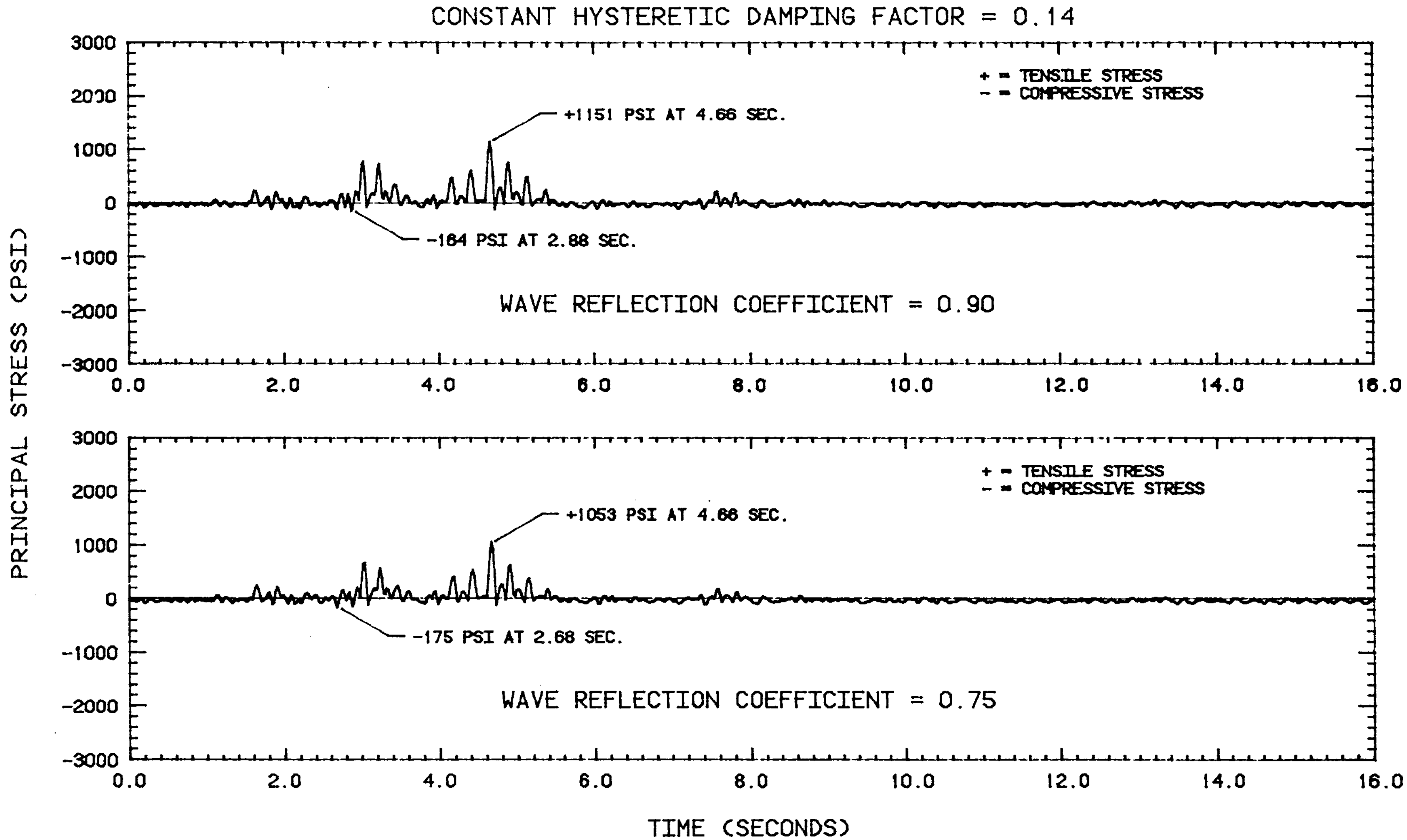


FIGURE 9-79 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 114 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

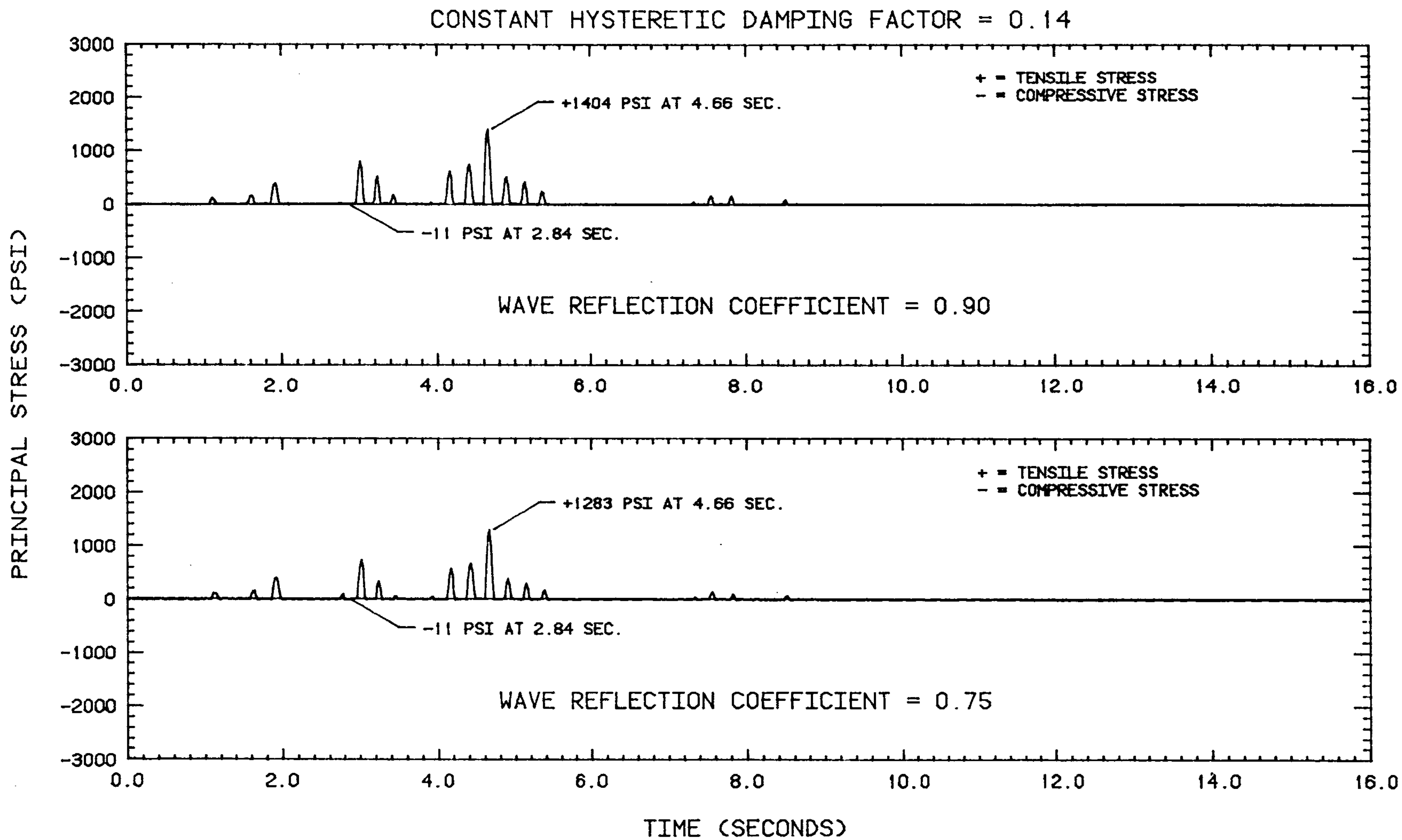


FIGURE 9-80 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 217 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

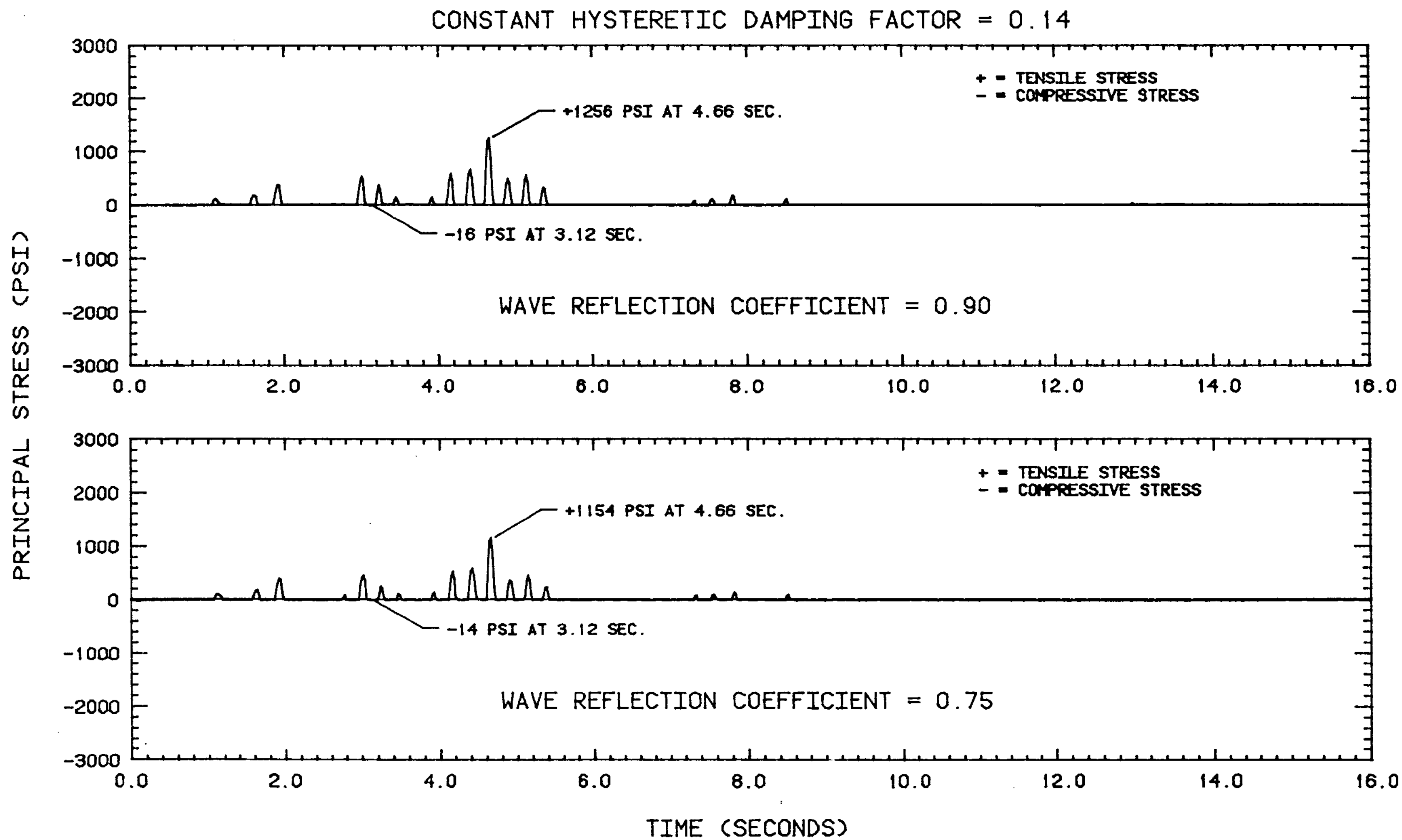


FIGURE 9-81 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 218 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

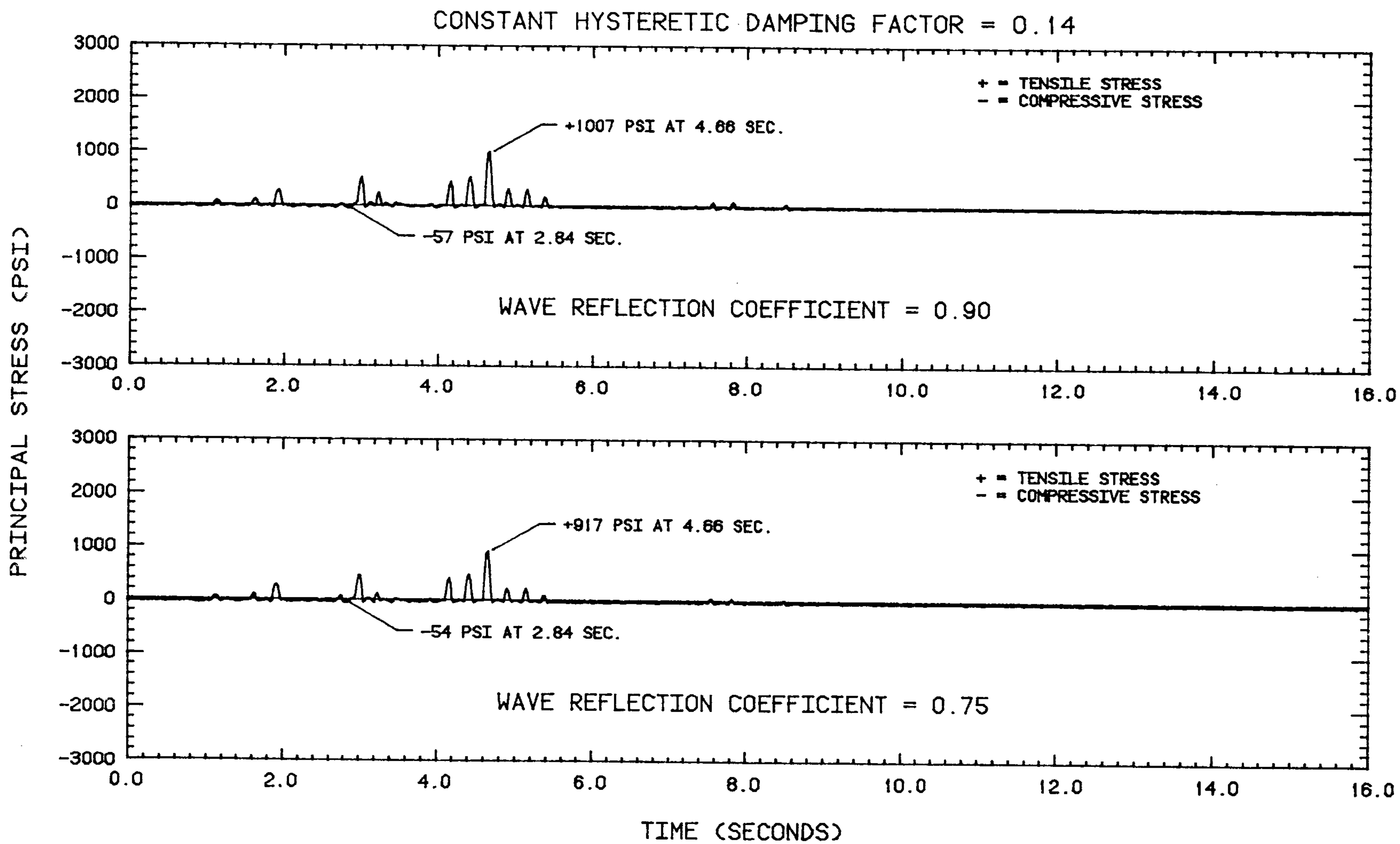


FIGURE 9-82 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 227 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

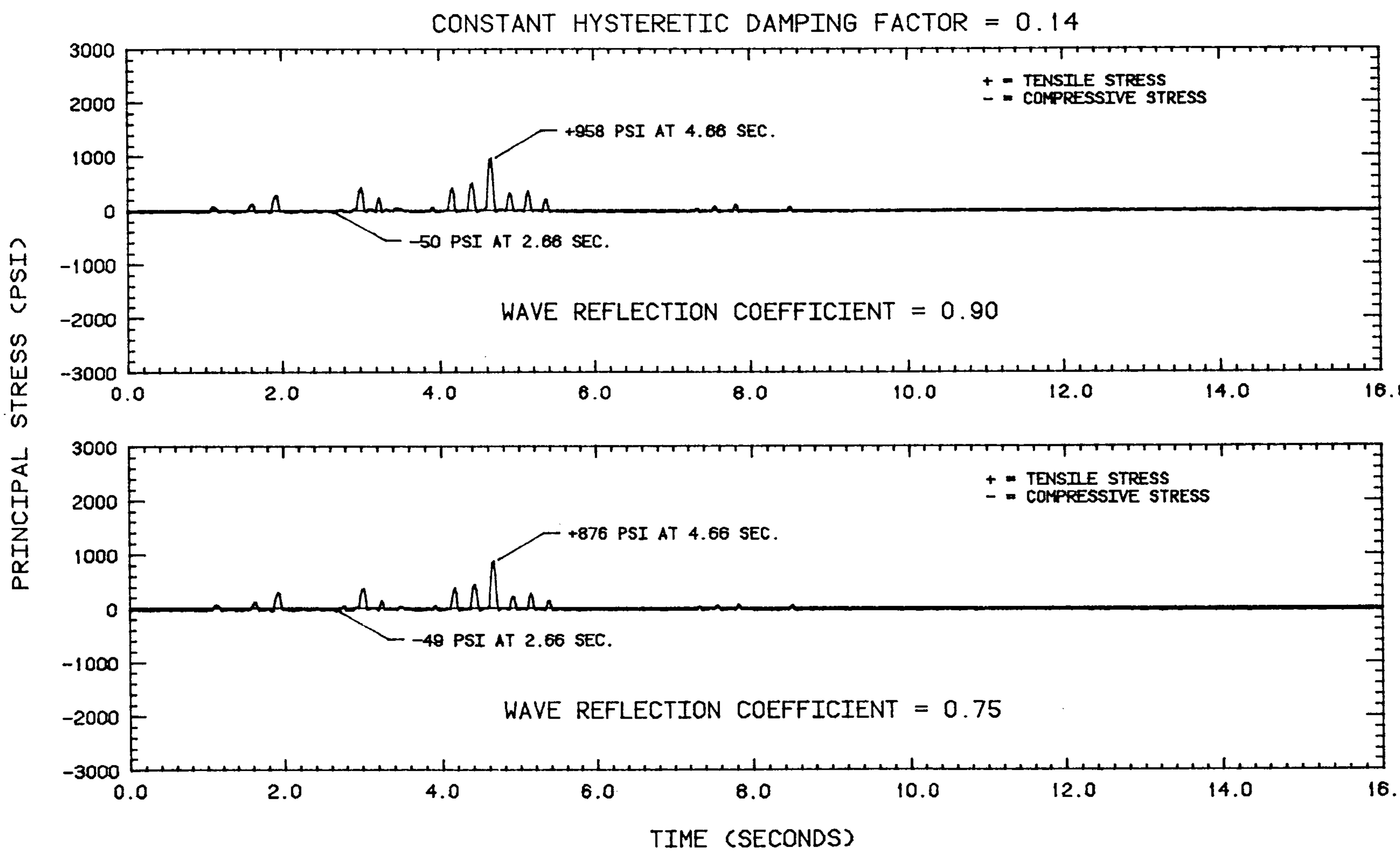


FIGURE 9-83 MAJOR PRINCIPAL STRESS RESPONSE AT NODAL POINT 228 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

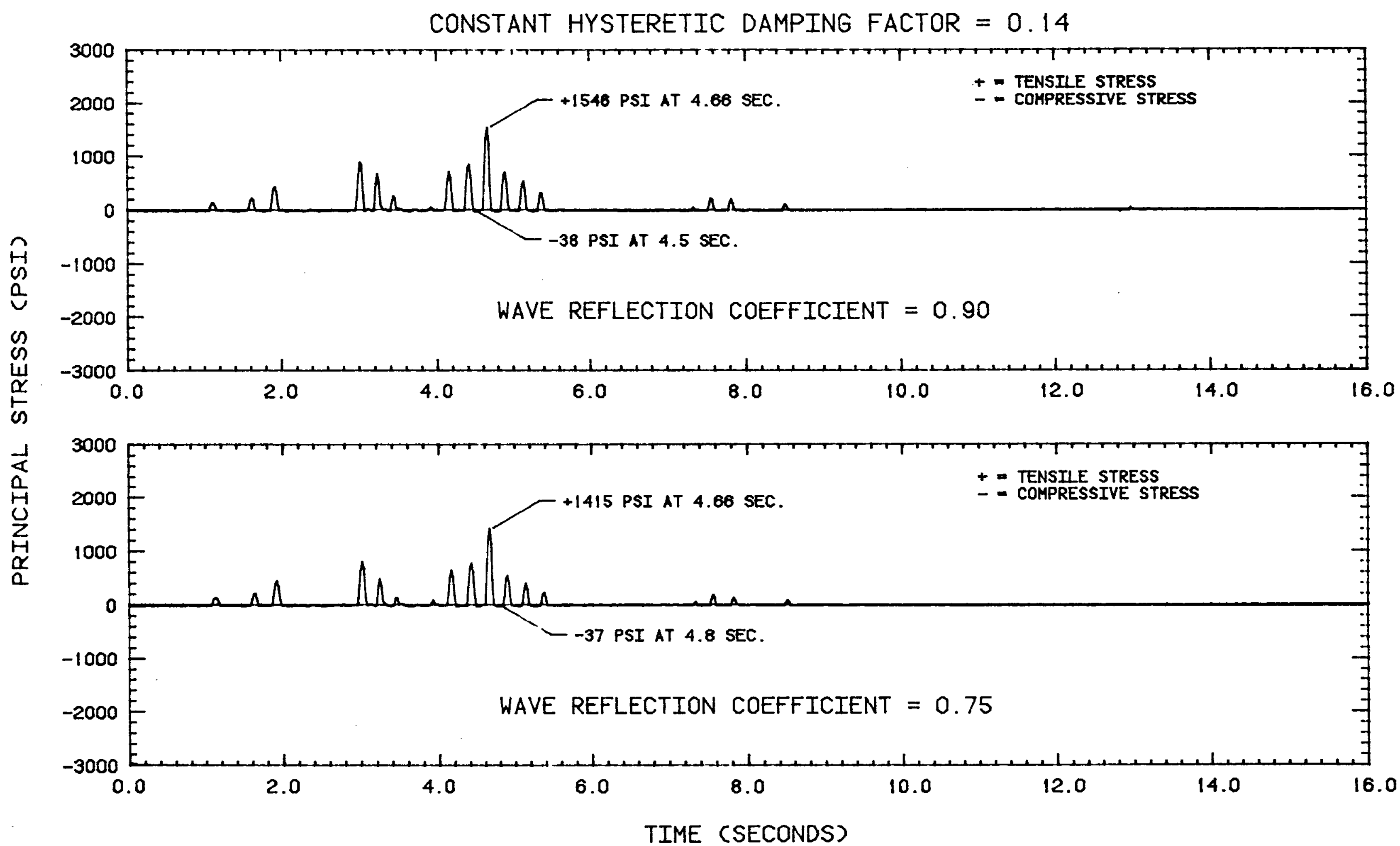


FIGURE 9-84 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

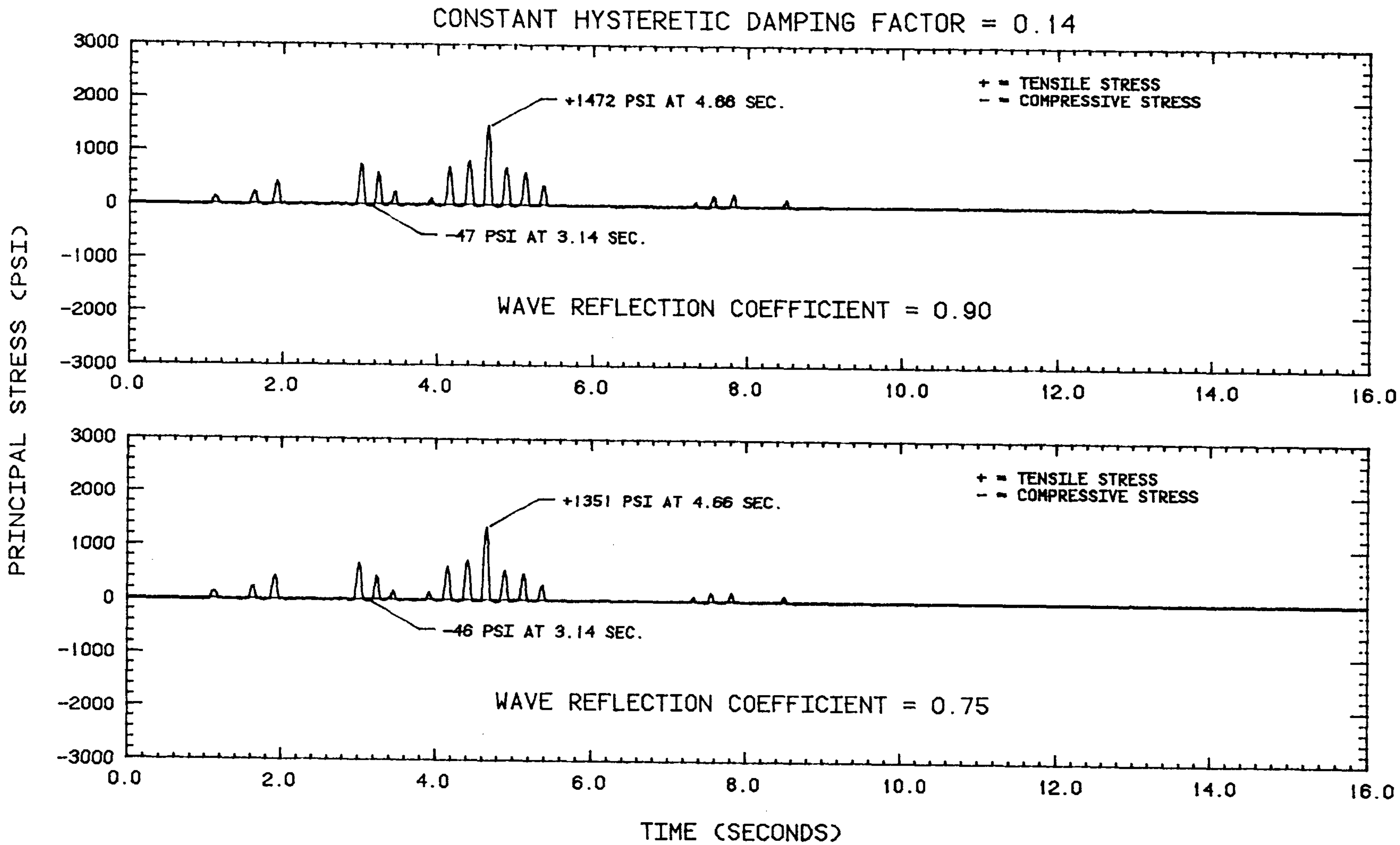


FIGURE 9-85 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 1 IN ELEMENT 9 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

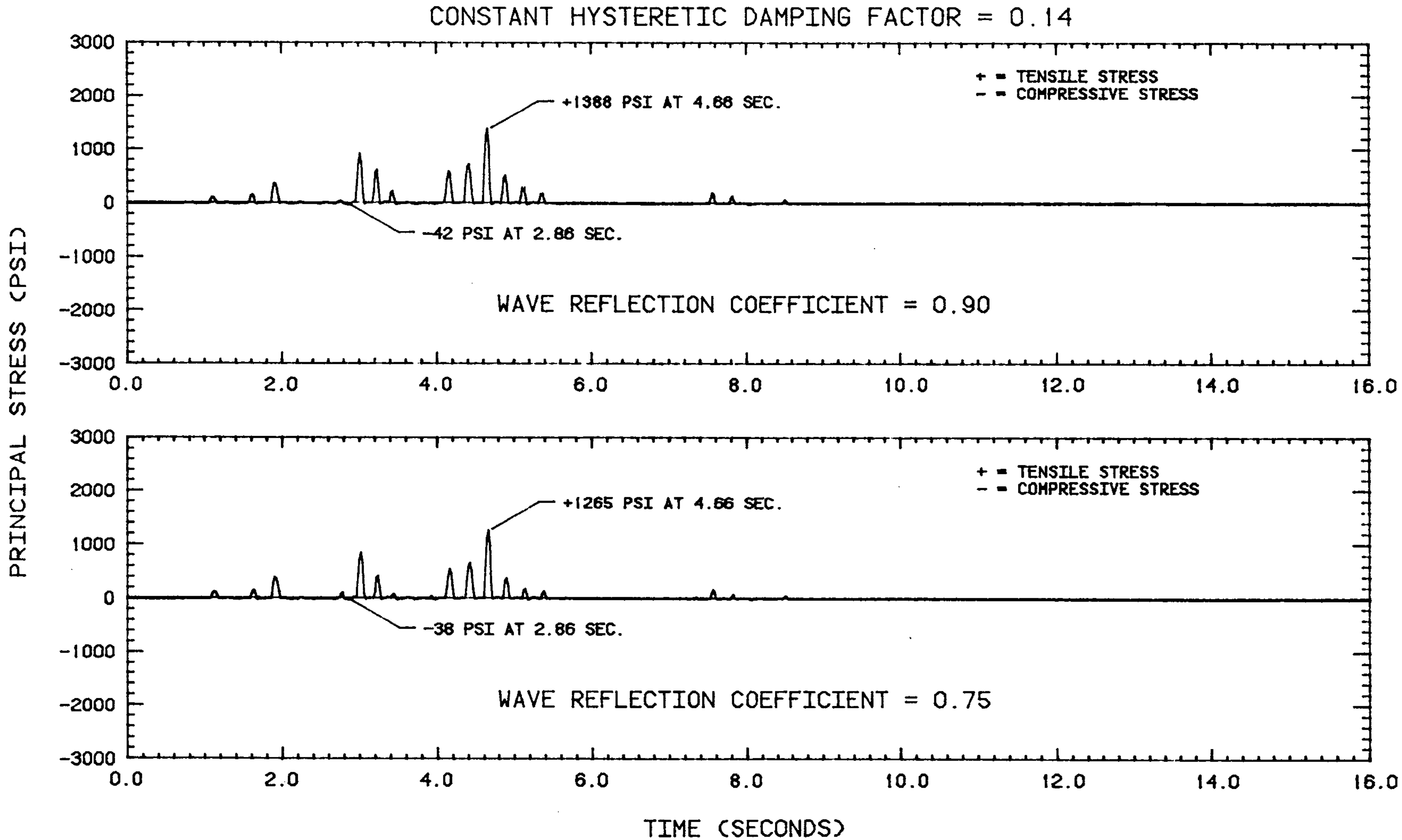


FIGURE 9-86 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 1 IN ELEMENT 8 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

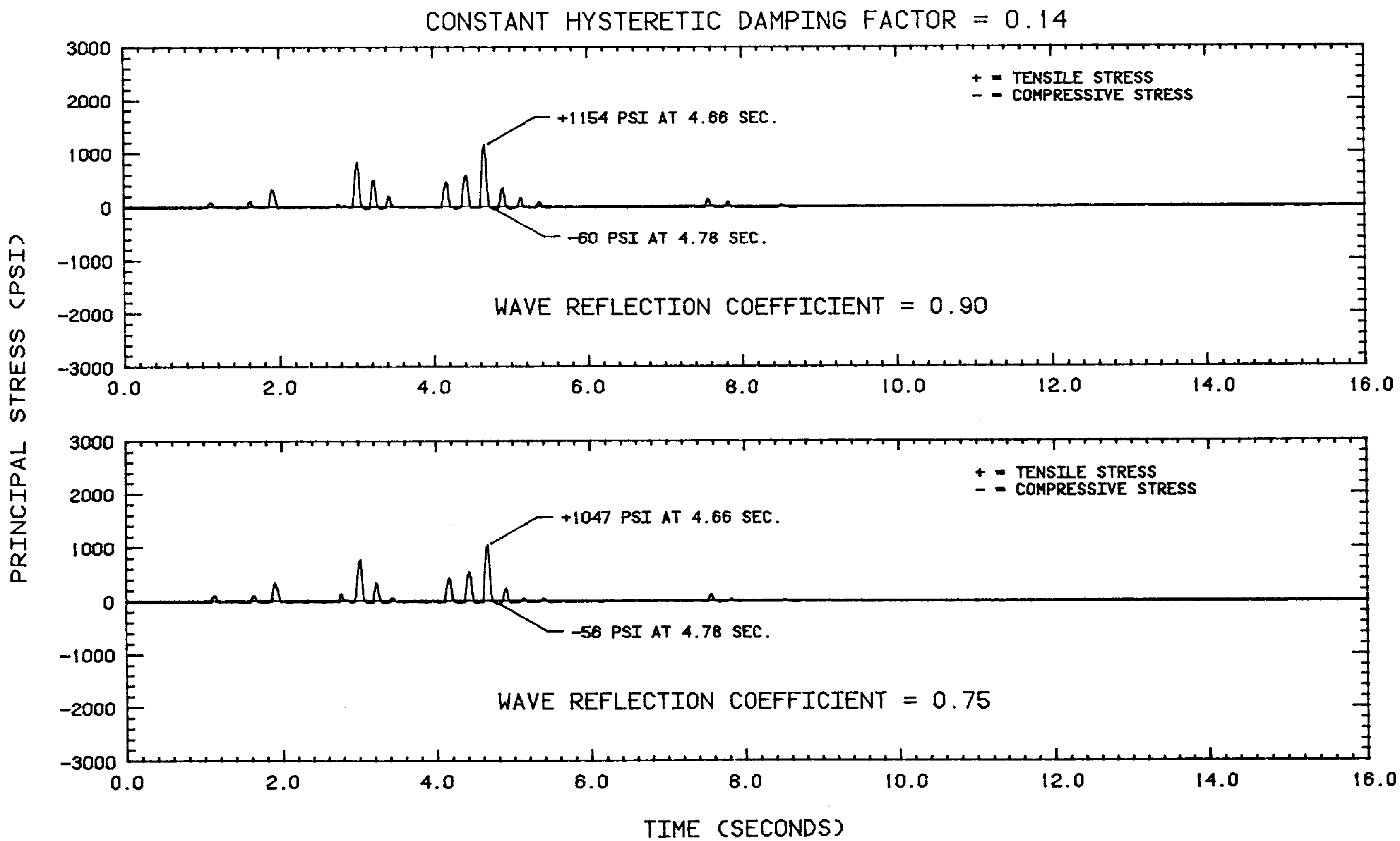


FIGURE 9-87 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 8 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

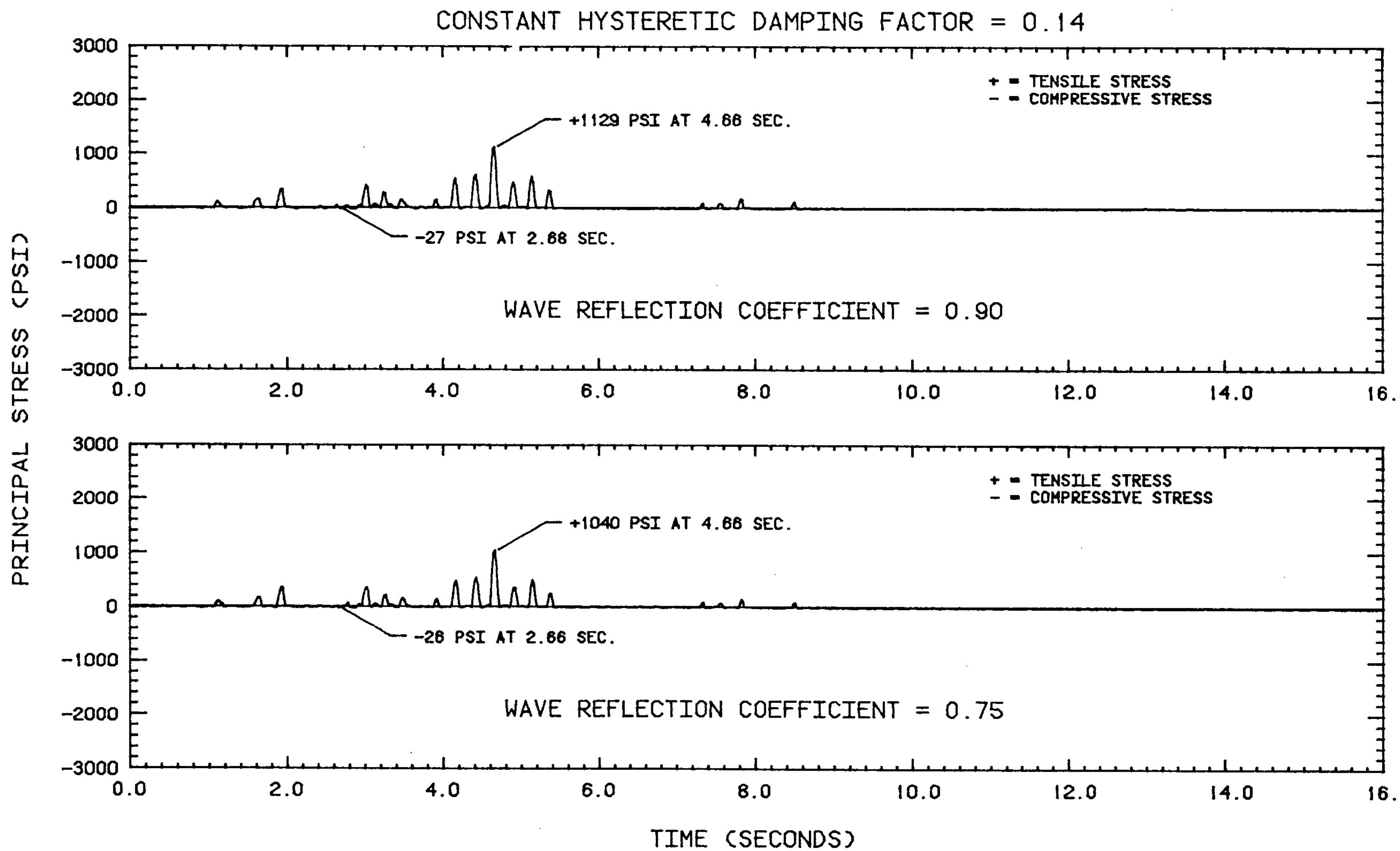


FIGURE 9-88 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 10 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

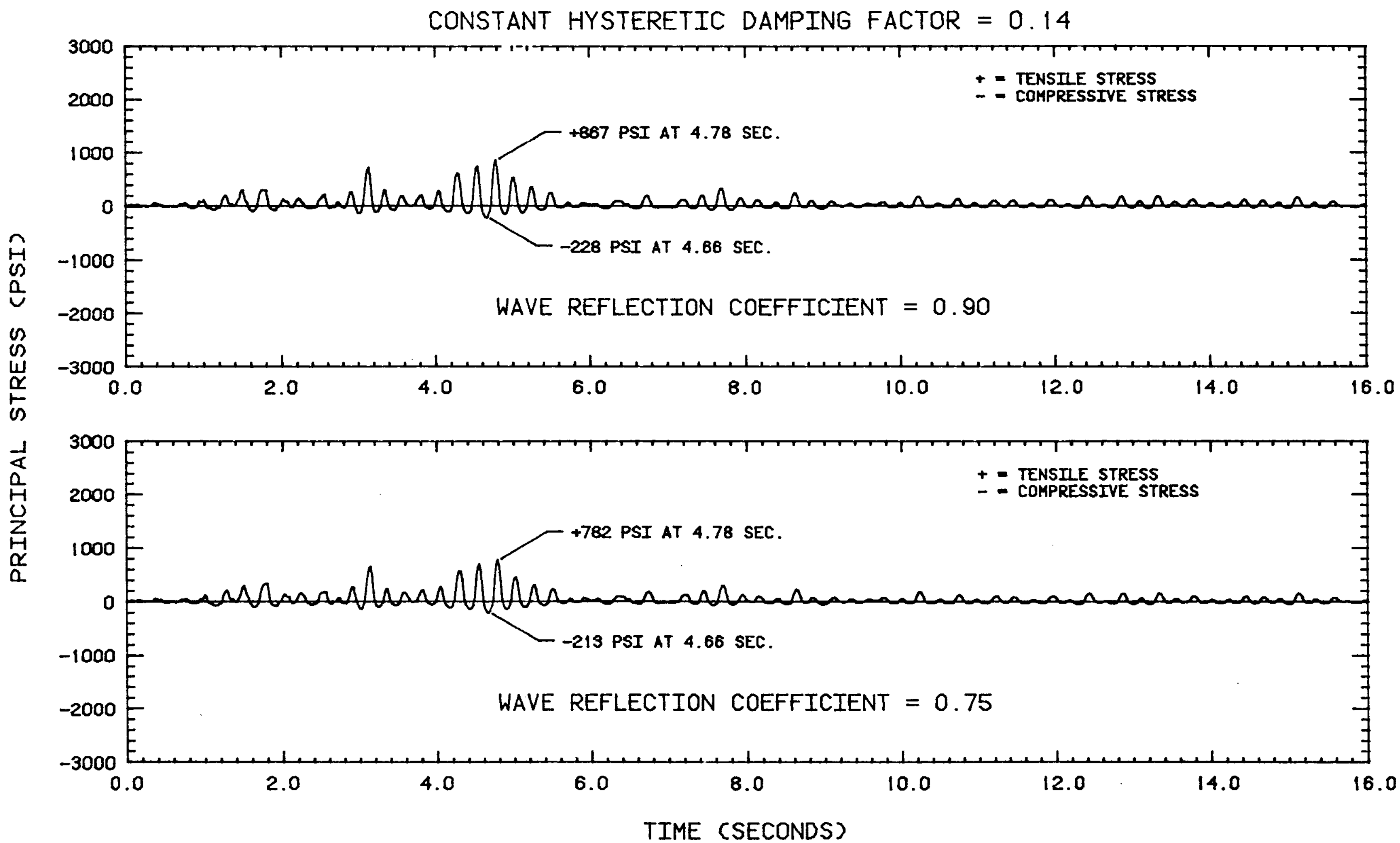


FIGURE 9-89 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 56 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

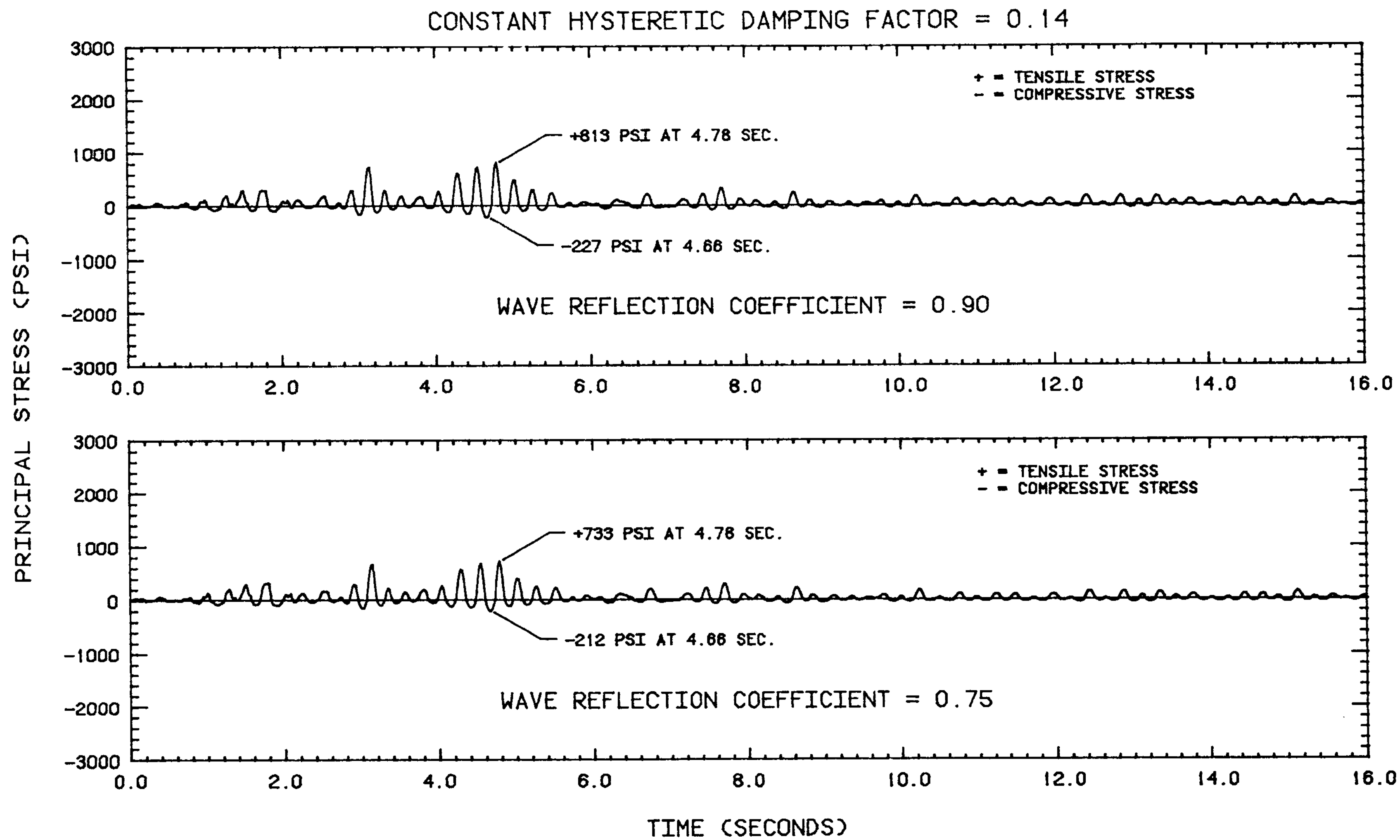


FIGURE 9-90 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 3 IN ELEMENT 56 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

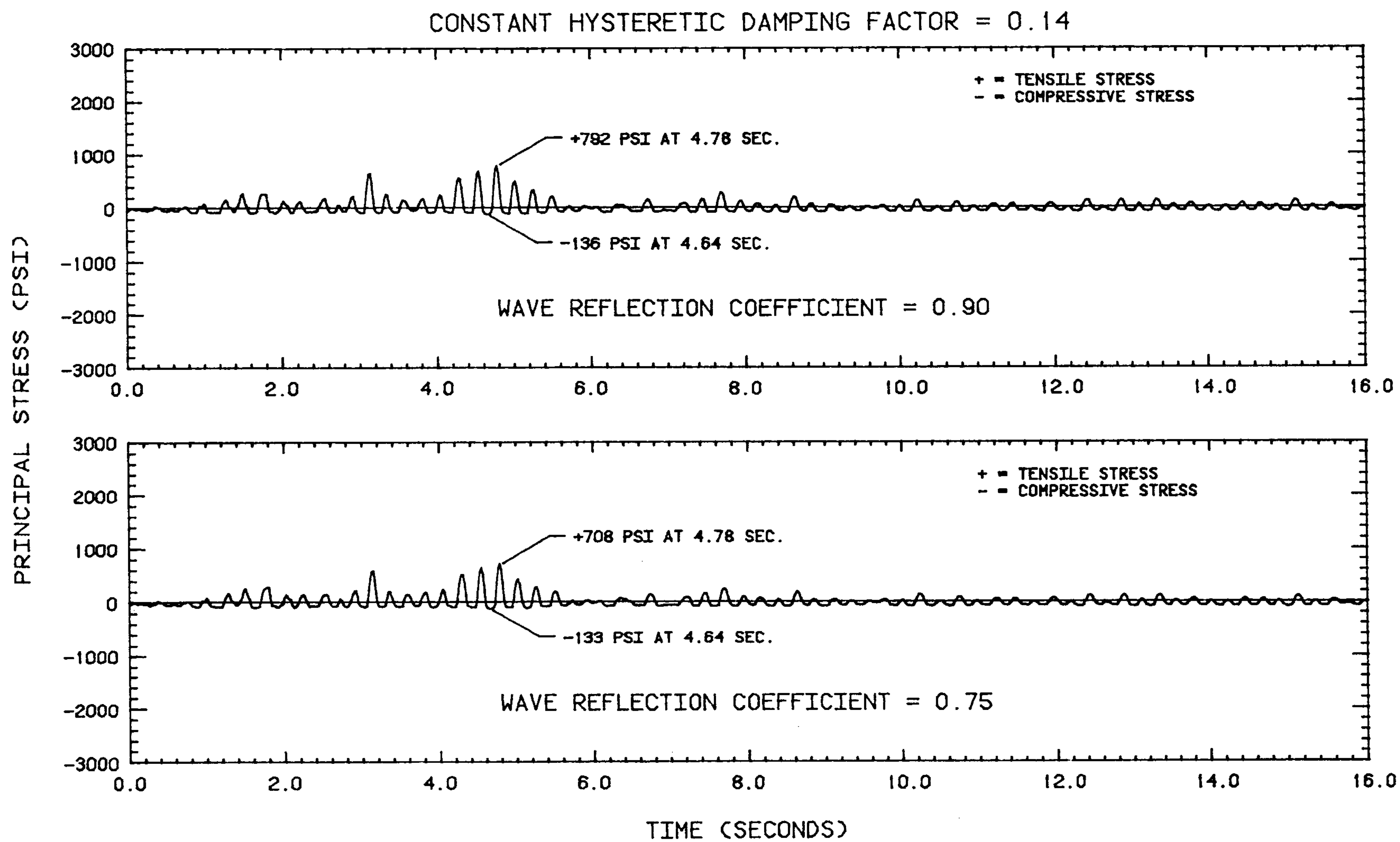


FIGURE 9-91 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 52 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

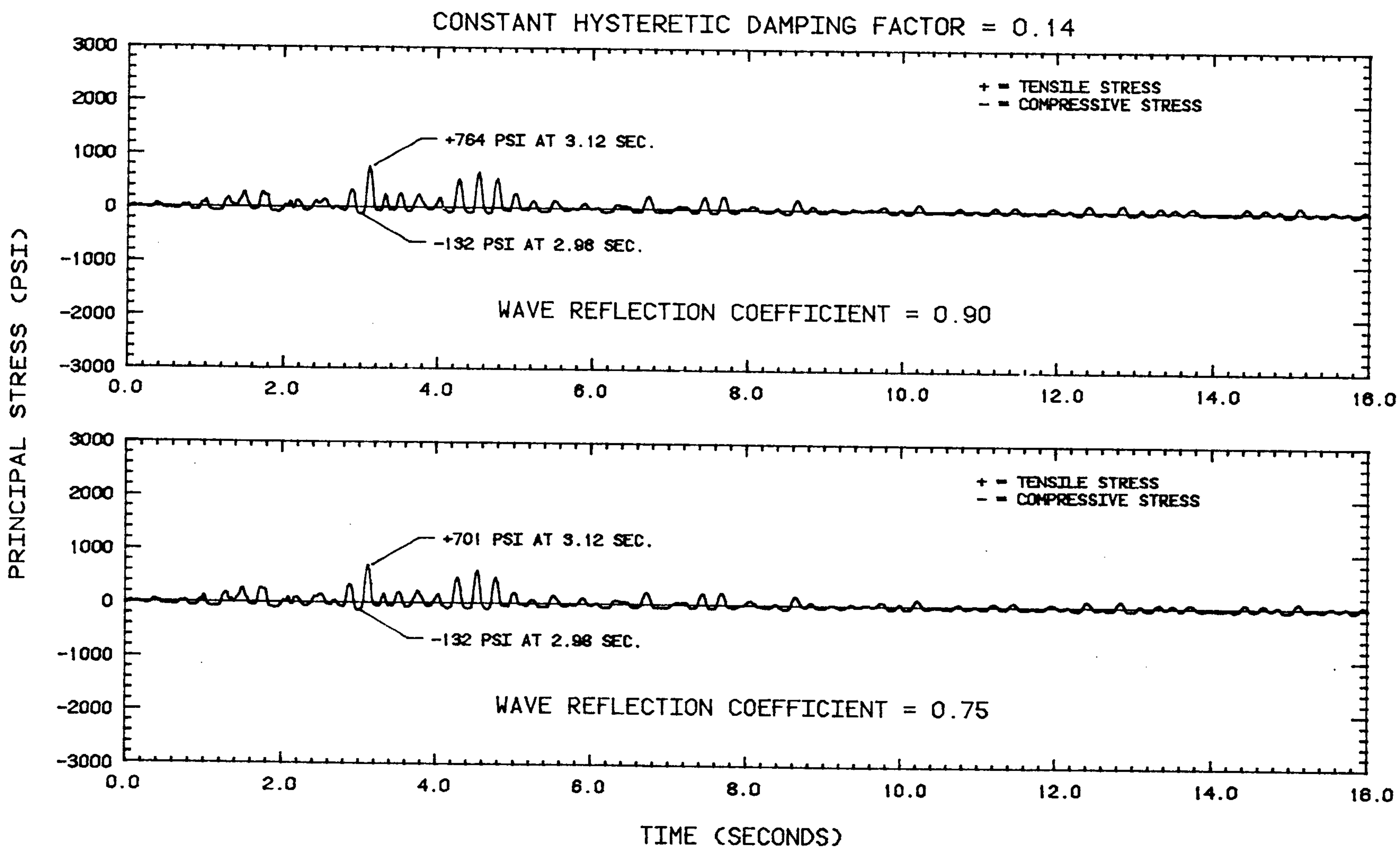


FIGURE 9-92 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 3 IN ELEMENT 51 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

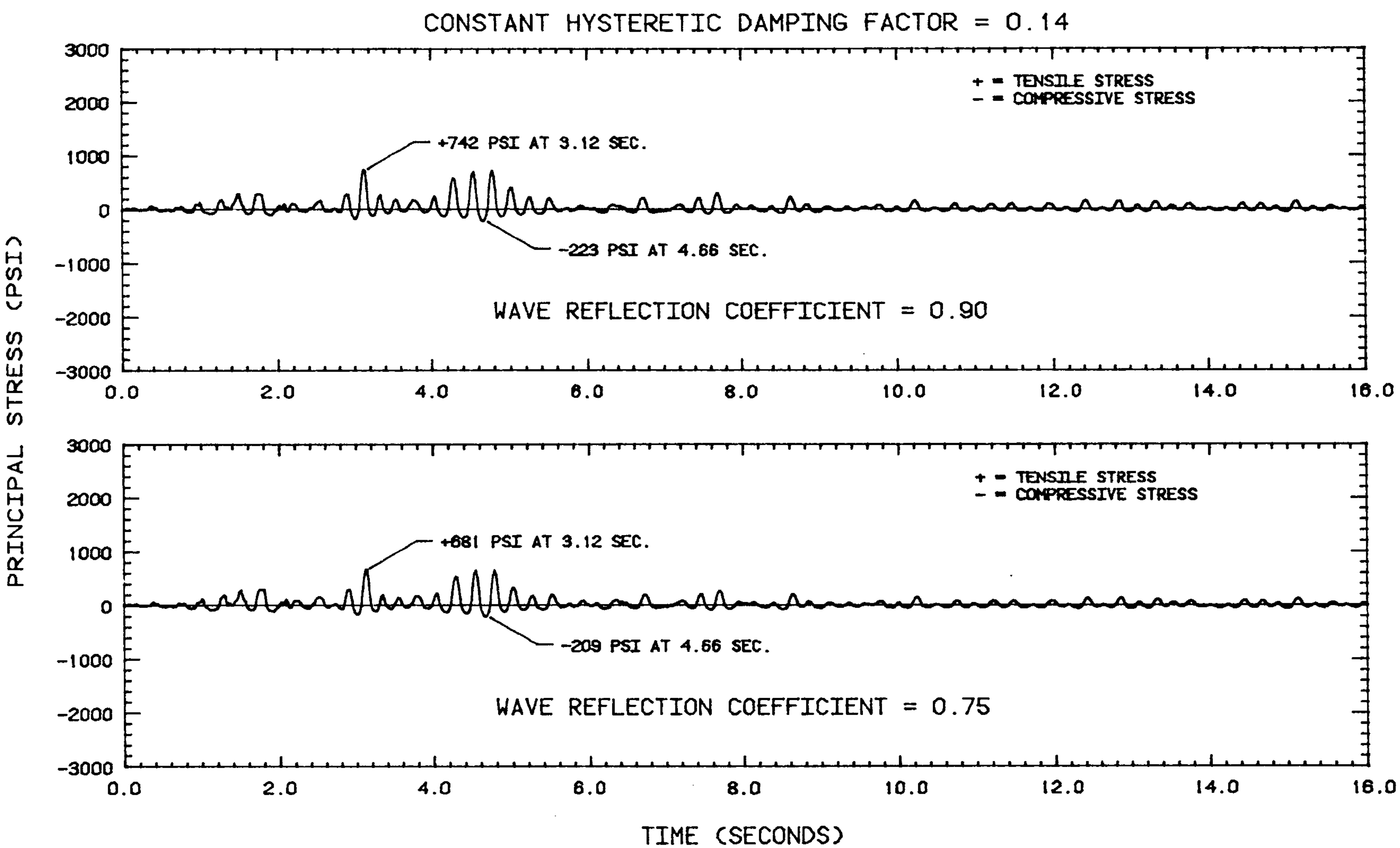


FIGURE 9-93 MAJOR PRINCIPAL STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 55 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

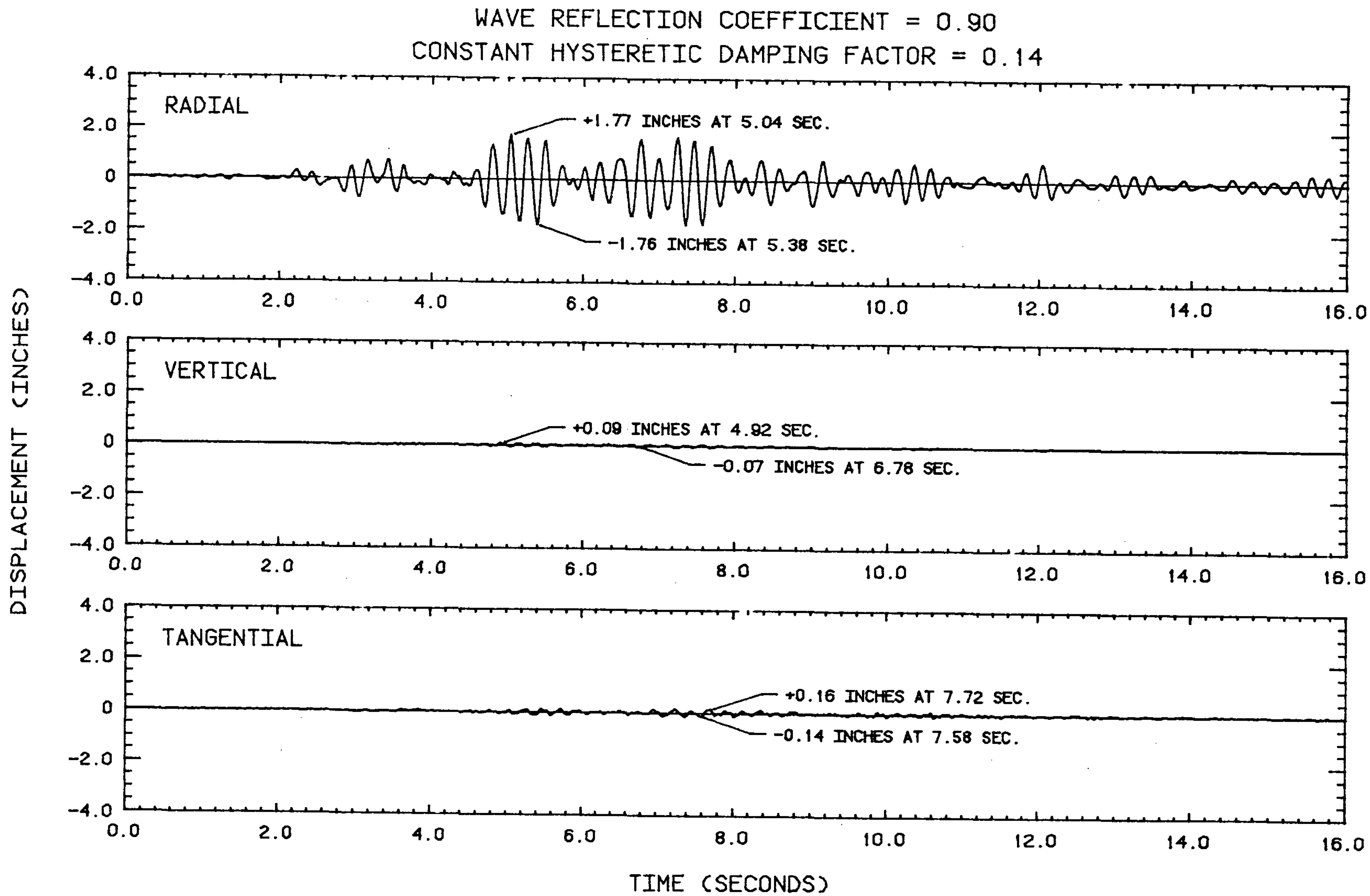


FIGURE 9-94 DYNAMIC DISPLACEMENT (RADIAL, VERTICAL, AND TANGENTIAL) RESPONSE AT DAM CREST NODAL POINT 303 DUE TO QUAKE 1. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET). STATIC EFFECTS ARE EXCLUDED.

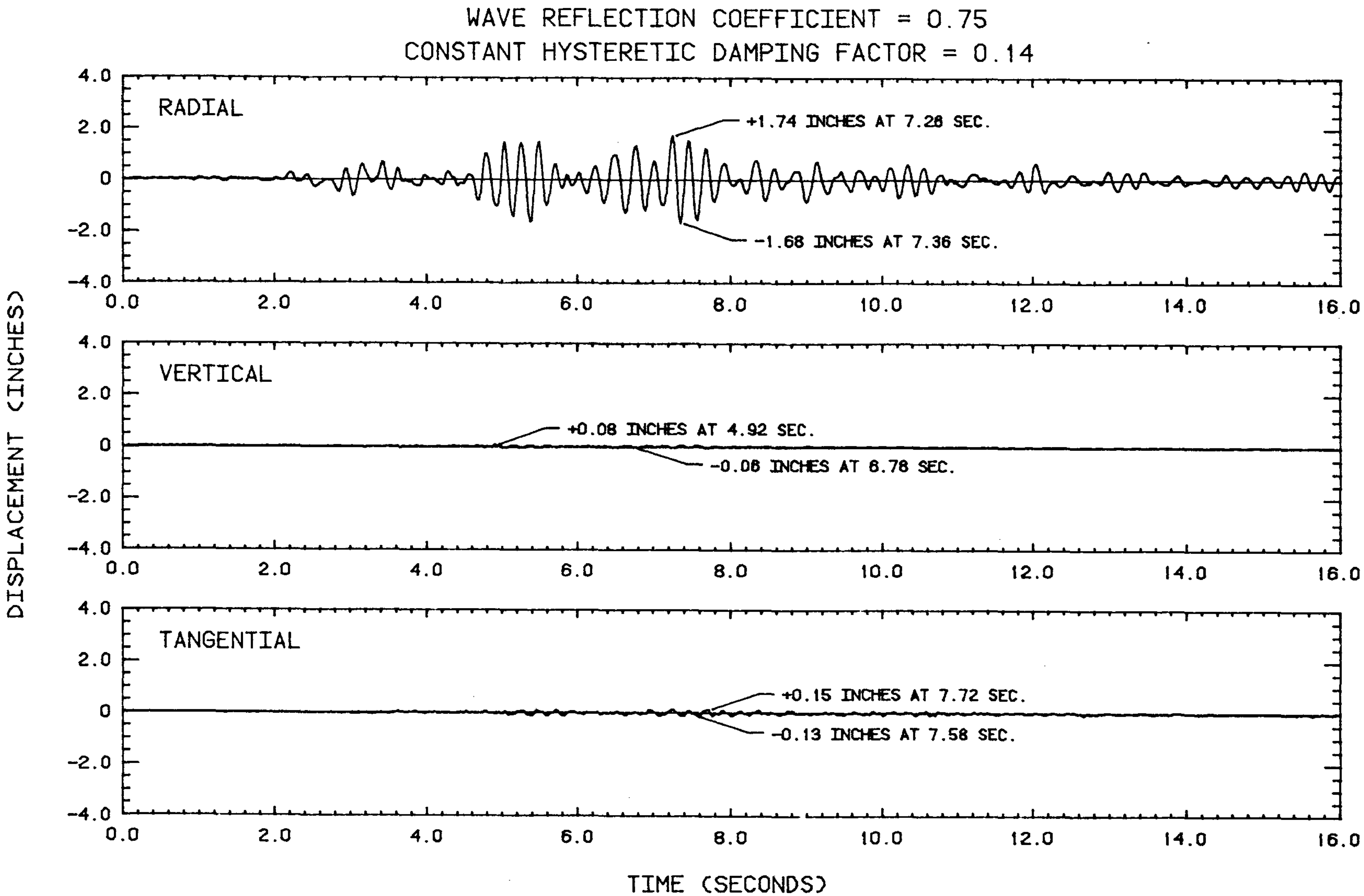


FIGURE 9-95 DYNAMIC DISPLACEMENT (RADIAL, VERTICAL, AND TANGENTIAL) RESPONSE AT DAM CREST NODAL POINT 303 DUE TO QUAKE 1. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET). STATIC EFFECTS ARE EXCLUDED.

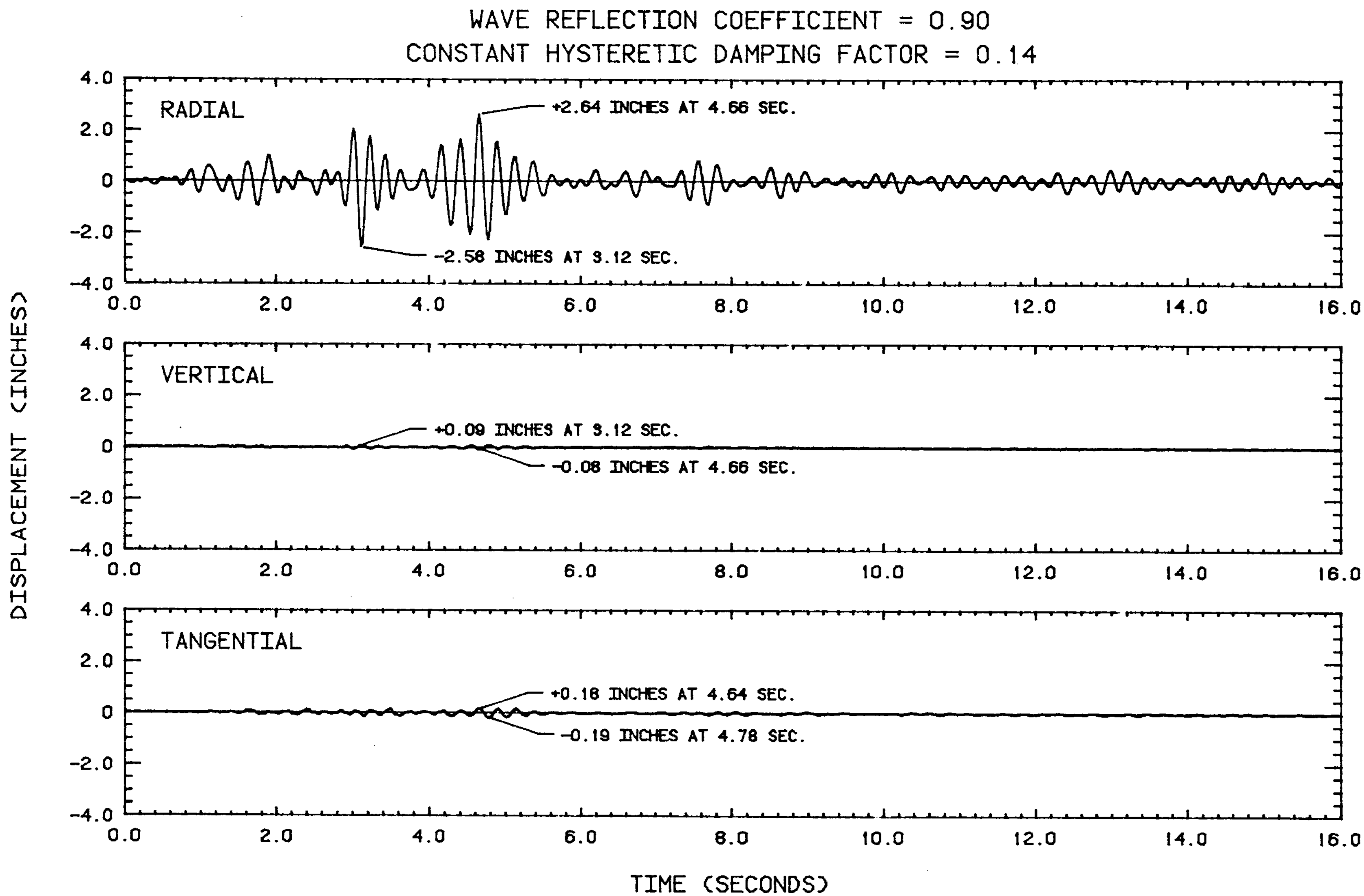


FIGURE 9-96 DYNAMIC DISPLACEMENT (RADIAL, VERTICAL, AND TANGENTIAL) RESPONSE AT DAM CREST NODAL POINT 303 DUE TO QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET). STATIC EFFECTS ARE EXCLUDED.

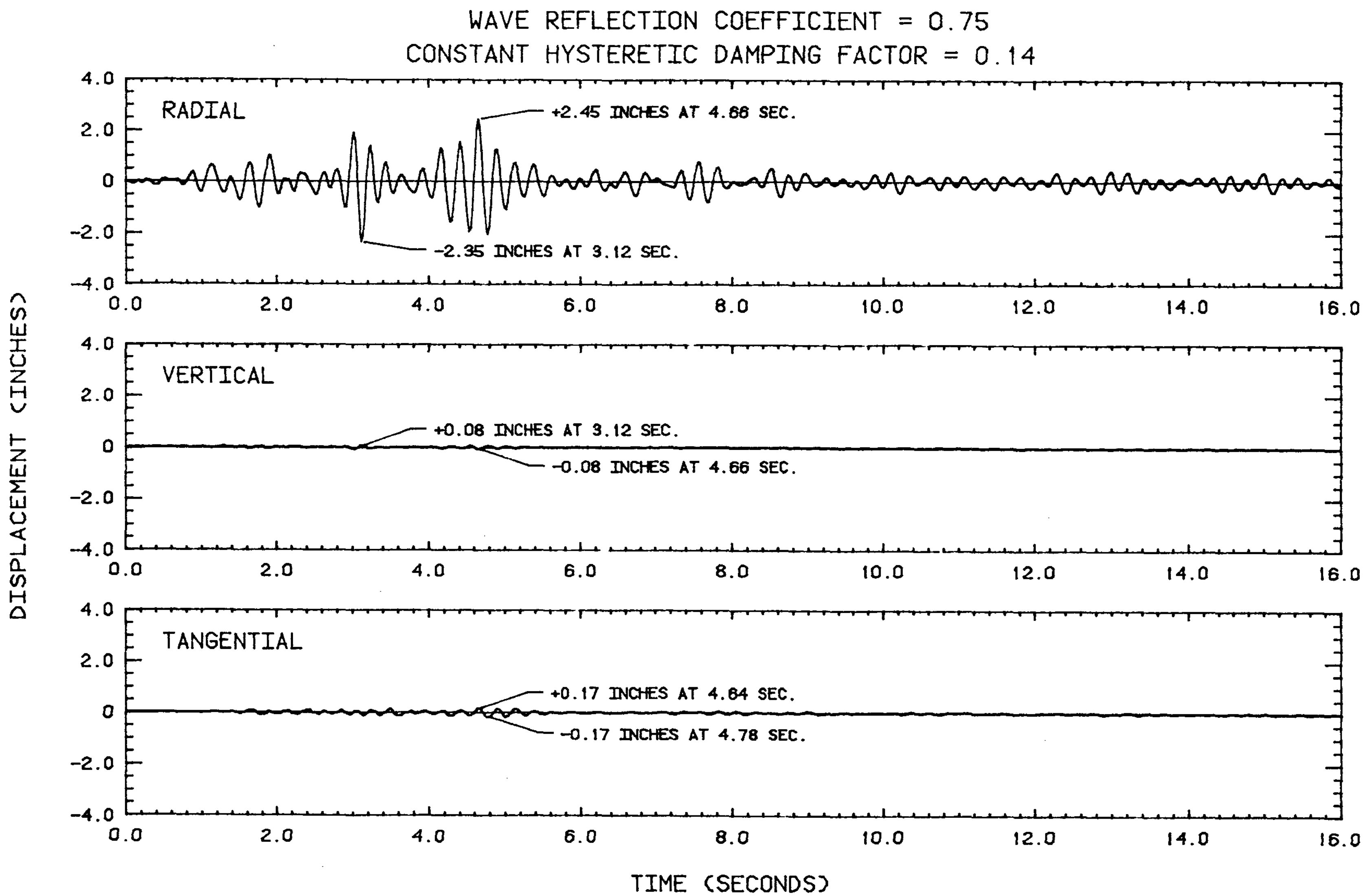


FIGURE 9-97 DYNAMIC DISPLACEMENT (RADIAL, VERTICAL, AND TANGENTIAL) RESPONSE AT DAM CREST NODAL POINT 303 DUE TO QUAKE 2. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET). STATIC EFFECTS ARE EXCLUDED.

APPENDIX A

SEISMICITY AND SEISMIC INTENSITY STUDY

SEISMICITY AND SEISMIC INTENSITY STUDY

ENGLEBRIGHT DAM, CALIFORNIA

for

CORPS OF ENGINEERS, SACRAMENTO

by

Bruce A. Bolt

Registered Geologist and Geophysicist (California)

and

Professor of Seismology

January, 1983

CONTENTS

I. Introduction

This report gives an analysis of the seismicity relevant to design considerations for Englebright Dam, Yuba and Nevada Counties, in northern California. The aim of the analysis is to provide recommendations on appropriate seismic parameters of design earthquakes which might be expected in the vicinity of Englebright Dam. The current requirements of the Corps of Engineers regarding selection of design earthquakes is to determine an appropriate maximum earthquake (ME) defined as the severest earthquake possible at the site, on the basis of the geological and seismological evidence. In addition, attention is given to an operating basis earthquake (OBE) selected on a probabilistic basis from the evidence as being the maximum likely to occur during the life of the project. It is understood that after selection in this report of the design earthquake parameters, these values will be applied in a subsequent study to select appropriate ground-motion time histories for the project.

Englebright Dam was completed in 1941 and is located on the Yuba River about 25 miles northeast of Marysville, California. The location of the reservoir is shown in Fig. 1. The dam is a concrete arch structure with a maximum height of 260 feet and a length of 1,142 feet. An evaluation report by the U.S. Army Engineer District, Sacramento, dated 1970, was consulted and it describes the dam reservoir and other facilities.

The present investigation was conducted along five lines:

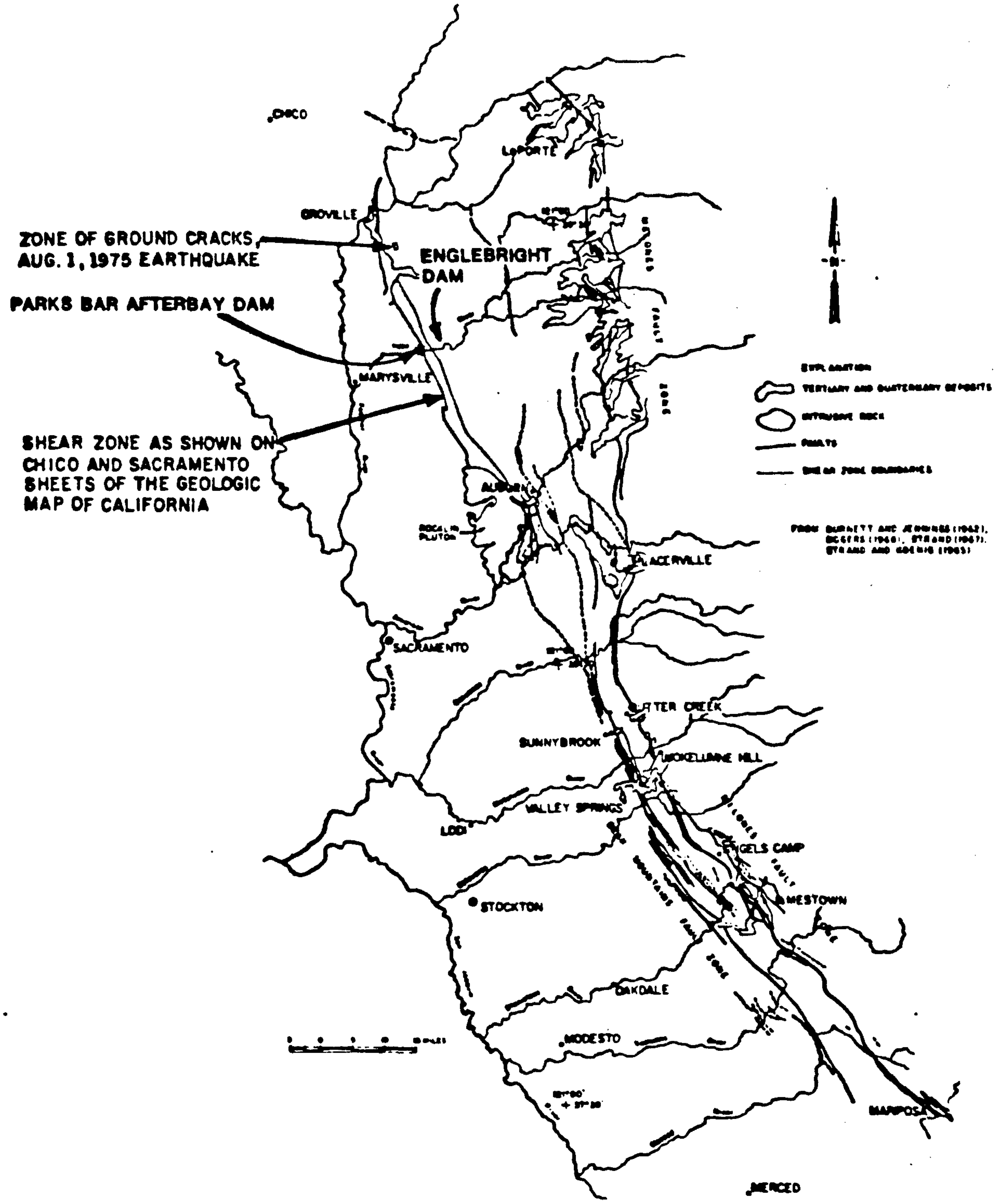

CHICO
OROVILLE
ZONE OF GROUND CRACKS
AUG. 1, 1975 EARTHQUAKE
ENGLEBRIGHT
DAM
PARKS BAR AFTERBAY DAM
MARYSVILLE
SHEAR ZONE AS SHOWN ON
CHICO AND SACRAMENTO
SHEETS OF THE GEOLOGIC
MAP OF CALIFORNIA
SACRAMENTO
SUNNYBROOK
MOKELUMNE HILL
VALLEY SPRINGS
LODI
STOCKTON
OAKDALE
MODESTO
MARIPOSA
MERCED

FIG. 1

a) Information on the regional and site geological structures was reviewed. In particular, the broad system of mapped faults which may be the source of damaging earthquakes in the region was examined together with evidence on regional strain and the activity of the significant faults in Quaternary times.

b) All available earthquake occurrence data for the region were gathered and reviewed. These included historical catalogs of felt reports and the modern instrumentally-based catalogs. This seismicity record was used to define areas of earthquake activity, to establish historic intensities in the region, and to indicate the likely rate of occurrence of the more intense earthquakes near the dam.

c) Based on the above regional tectonic and seismicity information, inferences were made on the likely location and size of earthquake sources in the region that might affect Englebright dam.

d) For each selected source, earthquake parameters of engineering importance were estimated for the expected strong ground shaking at the dam, both in terms of what is mechanically feasible and what might be probable during the life of the dam.

e) The recommended seismic intensity parameters were tabulated in a form suitable for the subsequent construction of seismograms of acceleration, velocity and displacement of the ground.

Attention has been given in the assessments of the parameters of the maximum earthquakes to the uncertainties in both the methods of estimation and the observational data. So far as can be determined, Englebright dam has been shaken in historical times by only one great earthquake, namely the 1906 San Francisco earthquake, produced on the

San Andreas fault about 190 km from the site. No significant earthquake of moderate size intensity has been known to occur in the last 150 years within 50 km of the site except the 1975 Oroville earthquake (magnitude 5.7) centered about 30 km to the northwest. We have thus little direct recent information on the distribution of seismic intensities near the dam. Since the construction of the dam, however, many special studies have been made about the regional geology and tectonic conditions, in particular for other engineered structures along the Foothill fault system. As well, for a short period in recent years detailed seismicity information has been forthcoming for the region. The basic models for the required intensity assessments are thus relatively firm. The confidence in the ground motion estimates is enhanced by crosschecking all the pieces of geological and seismological evidence and by allowing for the conditional probabilities and the estimated life of the project. The results presented here have also been crosschecked with estimates of similar parameters made independently in recent years in this region of northern California (see References).

II. Tectonic Environment

The geologic maps of the region (see Fig. 1) show that Englebright Dam is located in the western Sierra Nevada foothills. The geological formations in the area consist of a complex assemblage of metasedimentary and metavolcanic rocks. This section of the western Sierra Nevada metamorphic belt has a number of major lineaments and related structures called generally the Foothills fault system.

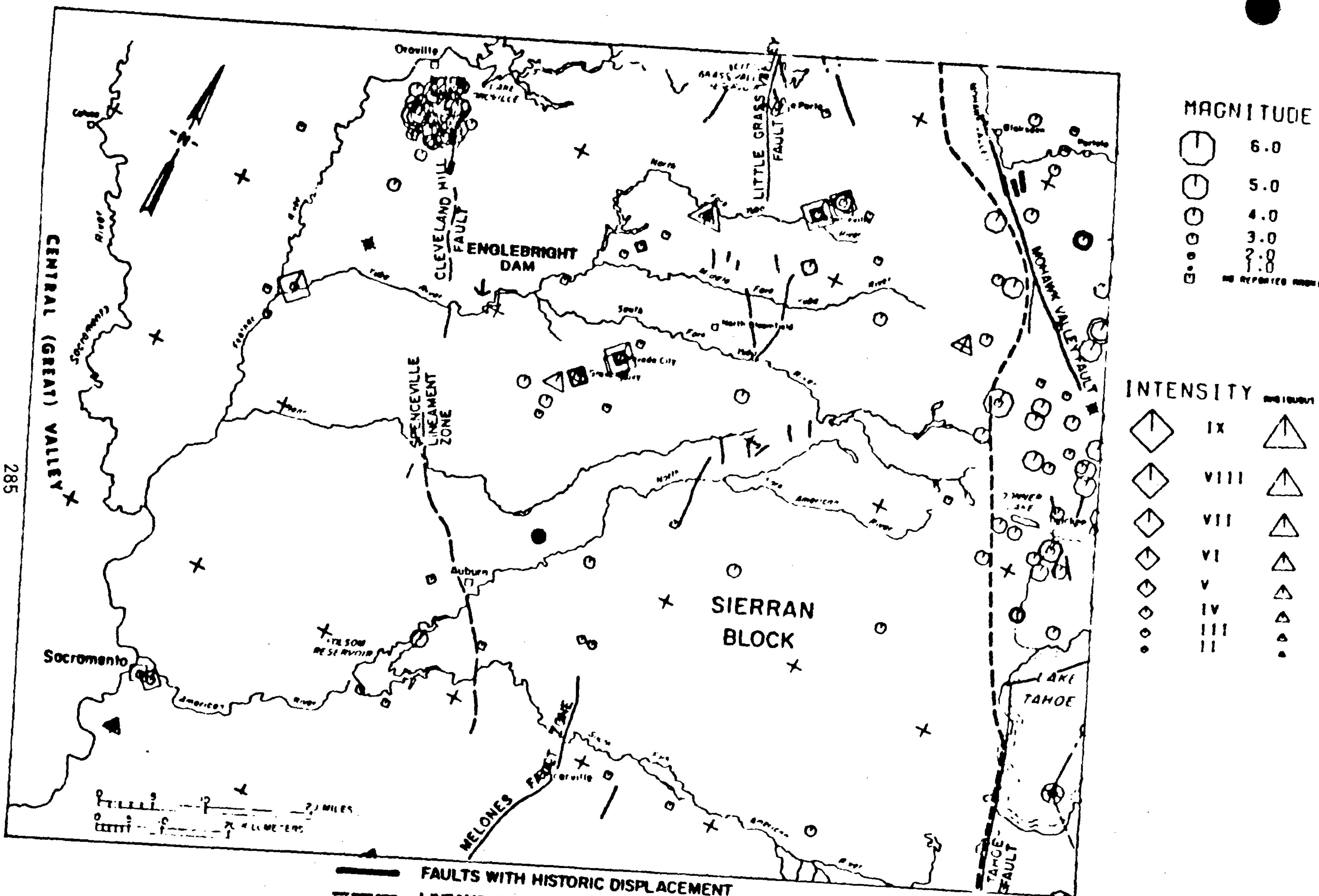
It is unnecessary here to give a detailed account of the regional structural geology because of the availability to the Corps of Engineers of recent thorough and up-to-date special reports. The first is an extensive study prepared for the U.S. Army Corps of Engineers by Woodward Clyde Consultants entitled "Evaluation for Potential for Earthquakes and Surface Faulting, Parks Bar Afterbay Dam, Yuba County, California" (1976). The Parks Bar Afterbay Dam is only 6 km from Englebright Dam and a considerable amount of the analysis carried out by Woodward Clyde Consultants applies to the present problem. In addition, there is a useful summary report by the Corps of Engineers entitled "Geologic Reconnaissance of the Englebright Dam and Lake Area" (1982). This report considers the regional geology, structural features, tectonics and seismicity of the region adjacent to Englebright Dam. A summary of the main relevant tectonic aspects of these studies is now given.

Englebright Dam is situated in the western Sierra Nevada metamorphic belt in a section identified as the Smartville ophiolite sequence. The sequence recurs as a folded and faulted block within the metamorphic belt and is northwest-trending and bordered to the west by the Browns Valley Ridge volcanics and to the east by the northern portion of the Sierra Nevada batholith. The metamorphic belt consists of a complex series of northwest-trending metasedimentary and metavolcanic rocks which range in age from Paleozoic to Jurassic, which are intruded in turn by a number of plutons generally of Mezozoic age. According to Clark (1976), the metamorphic belt has been subjected to folding, shearing and faulting, mostly at the end of the Jurassic period.

At Englebright Dam, the basement is metavolcanic rocks intruded by plutonic rocks ranging in composition from basic intrusives to diorites. The bedrock complex at the dam is mainly quartz diorite, metavolcanic rocks and diabase and metabasalt dikes. Available drill logs show that the river section, lower canyon slopes and much of the right abutment are underlain mainly by quartz diorite in the form of a massive crystalline rock, cut by numerous dikes. The left abutment, and part of the right abutment, is underlain predominately by metavolcanic rocks, such as meta-andesite and breccia that have consolidated by compression and metamorphism into a dense rock section. At the dam itself published field mapping shows that the rocks are highly fractured and jointed, with many shears and fault zones. Several narrow fault zones were encountered and described in the foundation explorations. There is no available evidence that any of these fault zones at the dam itself are extensive and have features that could be related to significant earthquakes.

From the point of view of seismic sources, the main structural feature near Englebright Dam within the metamorphic belt is the Foothills fault system. Considerable field work and structural analysis of this system has been carried out in connection with studies related to the Dam site near Auburn to the south and to the 1975 earthquake that occurred near Oroville Dam to the north (see Figure 1). The Foothills fault system has two main sub-systems, the Melones fault zone near its eastern edge and the Bear Mountain fault near its western edge. Englebright Dam is near the western portion of the Foothills fault system. In this section the fault shear zone has a strike with a similar northwest trend as that of the Bear Mountain fault zone although there is no direct geological evidence of a structural connection.

The regional shear zone of interest (see Figure 2 for simplified version) is located about 6-1/2 km west of Englebright Dam. The regional geology map (Plate 1) of the U.S. Corps of Engineers report gives the most recent details concerning the lineaments which make up the western part of the Foothills fault zone in this region. To the north is the Cleveland Hill fault which showed ground cracking after the August 1, 1975 Oroville earthquake. Ground rupture with displacement of about 5 cms occurred on the Cleveland Hill fault at that time. Strong but discontinuous topographic lineaments extend southward from the Cleveland Hill fault to the Spenceville lineament zone and on to the Bear River (see Figure 2). Between the Cleveland Hill fault and the Yuba River the zone as shown in Plate 1 has three predominant lineaments that have been named the Paynes Peak lineament, the Swain Ravine lineament and the Prairie Creek lineament. These lineaments continue south and the Prairie



FAULTS WITH HISTORIC DISPLACEMENT

LINEAMENT ZONES WITH EVIDENCE OF QUATERNARY DISPLACEMENT

TECTONIC DIVISION BOUNDARY

FIGURE 2

Creek and the Swain Ravine sections have been extended southwards of the Yuba River. Southward projection of these lineaments, past the Auburn Dam site to the south, finally encounters the Bear Mountain fault zone (see Figure 1), as mentioned in the last paragraph.

The main structural features of the main fault zone described above (to the west of Englebright Dam) are a lack of mechanical continuity between the Cleveland Hill fault and the point where the zone is truncated by the Rockland pluton, about 30 km south of Englebright Dam (see Figure 1). Despite a great deal of field work, no evidence has been found that extended fault rupture has occurred along this system. On the contrary, there are no extensive recent surface morphological features consistent with numerous fault slips with continuity over tens of km. (The upper bound to the length of fault available for rupture is the 45 km section from Oroville to the Rockland pluton.)

The intermittent and mechanically disjoint nature of the fault zone defined by the lineaments suggests that future seismic sources involving fault rupture in this zone will be limited in linear and vertical extent and similar to the offsets on the Cleveland Hill fault that produced the 1975 Oroville earthquake. It should be mentioned that both fault plane solutions and trenching across the ground cracks associated with the Oroville earthquake sequence confirm the predominant normal faulting mechanism for the Foothills fault system. Multiple fault displacements of the soil layers were observed with overall vertical motions of 4 to 5 cms. There was an indication from the nature of these offsets that the dip-slip observed occurred in at least three episodes of faulting during the last 5,000 to 100,000 years.

We must now consider more distant sources of earthquakes that might affect the site. Fifteen km to the east of Englebright Dam the State geological map (Jennings, 1976) and Plate 1 (Corps of Engineers, 1982) show an unnamed northwest-trending fault which passes through Willows Bar Reservoir. The total mapped length is about 70 km. Detailed geological discussions of the characteristics of this fault are not available but it can be considered the western boundary of the Melones fault zone mentioned earlier. Still further to the northeast is the Little Grass Valley (Dogwood Peak) fault. As mapped (see Figure 2), this extends about 40 km from the north fork of the Yuba River through Little Grass Valley. The trace has been defined by alignments of side hill ridges, topographic depressions and steps, and drainage offsets. It is reported that Pliocene age rocks are displaced approximately 200 meters across the fault, with the east side down. Further to the east, the active Mohawk Valley fault (see Figure 2), situated northeast of Lake Tahoe, must be considered because some geological field evidence exists for surface rupture on this fault, perhaps associated with the 1875 earthquake in that vicinity (maximum epicentral intensity VIII). The strike of the Mohawk Valley fault is towards Lake Tahoe and intersects an area in which short faults are mapped as having Quaternary fault displacement without any historic record of surface offsets. This source region to the east and the northeast is at a distance exceeding 80 km from the Englebright Dam.

Finally, mention must be made of the effect of a repetition of a 1906 earthquake due to rupture on the San Andreas fault to the west of the Englebright Dam. The San Andreas fault is about 200 km from the dam site and the intensity reported in 1906 in the vicinity was only IV to V on the Modified Mercalli Scale.

III. Seismicity of the Region

A complete listing of the seismicity for the region since the historical records began was obtained using the computer program RETRIEVE at the Seismographic Station, University of California, Berkeley. The list contains all relevant historical information from the Townley and Allen catalog (1939) and instrumental locations of earthquakes given in the Bulletin of the Seismographic Stations up to 1980 (see Bolt and Miller, 1971). A subset of this data for distances within 130 km of the site for the period 6/6/1934 to 3/31/1982 is attached to this report as Appendix 1. The subset is of particular value because after 1934 instrumental locations were made with an attempt at uniformity using the existing seismographic stations of the University of California. The earlier historical data are based on intensities for the region, are sparse and are very likely incomplete especially for parts of northern California away from well populated areas. For this reason the information of the nineteenth century is of limited value for the region. An important reassessment of the historical earthquake information has been given recently by Real, Toppazada and Parke (1978) and this catalog was also consulted. In addition, a summary of the seismicity affecting the region around the project may be found in the Woodward Clyde Consultants report on the "Parks Bar Afterbay Dam" for the U.S. Army Corps of Engineers (1976). Plots of the significant seismicity from the various sources are given in Figure 2.

The earthquake catalogs show that the adjacent region along the western foothills of the Sierra Nevada is of low seismicity, but nevertheless small earthquakes occur on an infrequent basis.

In referring to the seismicity lists (Appendix A) and the seismicity maps (see Figure 2) it should be understood that many intensities from historical earthquakes may refer to earthquakes with fault rupture sources well away from the vicinity of Englebright Dam and are merely felt or damage reports from the town or village within the immediate geographical region.

With this in mind, since 1850 there have been about 10 historical earthquakes having intensities MM VI to VIII within 60 km of Englebright Dam, but in no case is there any identification of Holocene faulting along the western Foothills fault system associated with any of these pre-1934 earthquakes. A number of these seismic intensities were reported from the Grass Valley-Nevada City area and from Downieville (see Figure 2).

The closest moderate size earthquakes to have occurred within 100 km of Englebright Dam since the instrumental record began are listed in Appendix A. Of these, the only ones of magnitude 5 or over that occurred within 50 km were part of the Oroville earthquake sequence of August, 1975 in which the main shock had Richter magnitude 5.7. As already mentioned in Section II, the earthquake was produced by rupture of the Cleveland Hill fault at a distance of about 30 km northwest of Englebright Dam. The fault mechanism was oblique slip, with both normal and right-lateral components and displacements. The focal depth of the main shock was approximately 5 km and the faulting was normal, with the west side down with a dip of about 60°.

In summary, few earthquakes of even small magnitude have been detected in the last few decades along adjacent segments of the Melones and Bear Mountain fault zones by sensitive seismograph stations at

Oroville and Jamestown. Some special installations of stations near the Rockland pluton (see Figure 1) about 50 km to the south have indicated a number of small earthquakes and microearthquakes (magnitude of 3 and less) in the vicinity of the pluton. As Figure 2 demonstrates, scrutiny of the whole historical record, going back to 1850, or about 130 years, leads to the conclusion that while the Foothills fault system should be regarded as seismically active, the occurrence rate of earthquakes is low and that the size of earthquakes is limited along it. The typical rate is probably close to the observation that no more than 2 to 5 earthquakes of magnitude 5 to 6 occurred in the northern part of the Foothills fault system in 130 years. The greatest instrumentally determined magnitude to be determined in the same tectonic region in 50 years was 5.7 for the 1975 Oroville earthquake.

IV. Effective Seismic Sources

The joint analyses of tectonic and seismicity information in Sections II and III leads to the definition of the fault systems that are capable of producing significant strong ground motion at Englebright Dam.

Of the various relevant fault systems discussed in Section II we find that a set of four only are quite adequate to cover all reasonable probabilities of significant intensities. These cases are set out in Table 1.

First, the Cleveland Hills fault (Figure 1) is taken to represent the western Foothills zone of tectonic lineaments shown in Figure 1. More explicitly, on the side of assessing the larger plausible intensities we identify this seismic source with the Swain Ravine lineament at its section of closest approach to Englebright Dam. (Detail is available in Plate 1, Corps of Engineers, 1982). Faulting on such a section is likely to be predominately normal dip-slip. A plausible length of rupture is taken as 25 km or about half the total distance along the fault zone from Oroville to the Rocklin Pluton. Such fault rupture lengths, accompanied by 20 to 30 cm of fault slip have been correlated with actual earthquakes of magnitude 6.0 to 6.25 (see Bonilla, 1970). Although such estimates have considerable uncertainty the numerical values taken above are all higher than might be expected on the average.

Secondly, the Little Grass Valley fault (Figure 2) is adopted as a section of the Melones fault zone (to the east of Englebright Dam) where a significant earthquake source might plausibly occur. In Table 1, the fault rupture is assumed to be centered 25 km from the dam and a 15-km

length is assumed to rupture in one dislocation. By a similar argument as above, i.e. correlation with similar earthquakes studied in the field, the event would correspond to an earthquake with magnitude up to about 6.0. If a portion of the Melones fault zone, nearer to Englebright Dam (see Figure 2) than the zone in Table 1 contained the seismic source (on the evidence discussed earlier a highly unlikely event), then the case would approach that already accounted for the Cleveland Hills fault.

For completeness, two more remote seismic sources are listed in Table 1. Both the Mohawk Valley fault and the San Andreas fault are capable of producing moderate to large earthquakes in the next few decades so that the resulting ground motion at Englebright Dam must be considered. At distances of 90 km and 200 km, respectively, however, even the maximum size earthquakes mechanically feasible on these faults would produce very small seismic wave amplitudes at the dam. The numerical estimates of these wave parameters are discussed in the next section (see Table 2).

Table 1

Major Regional Faults and Significance for Strong Motion

Earthquake Generation

Name	Distance from Dam (km)	Type	Length (km)	Activity in Project Life	Recurrence Interval
Cleveland Hills fault (Swain Ravine Lineament)	6.5	Normal	25	Yes	Holocene (historic)
Little Grass Valley fault (Melones fault zone)	25	Normal	15	?	Quaternary
Mohawk Valley fault	90	Dip-slip	50	Yes	Holocene
San Andreas fault	200	Strike-slip	1000	Yes	Historic

V. Intensity Parameters Near the Site

The discussion of the tectonics, seismicity and faults in the region set out in the previous sections now permit estimates to be made of appropriate ground motion intensities likely near Englebright Dam. These assessments are to be made within the terms of reference set out in Section I.

In the first place, the arguments have greatly reduced the number of fault sources most likely to give significant shaking at Englebright Dam. By a process of elimination, summarized in Table 1, we have arrived at four seismic sources for consideration as listed in Table 2. They are, in order of distance from the dam: Cleveland Hills fault, Little Grass Valley fault, Mohawk Valley fault and the San Andreas fault. The evidence is that significant earthquakes may occur along each of these faults in the next 50 to 100 years.

For the nearer faults, the strongest evidence is for activity on the Cleveland Hills fault or a related extension to the south (see the last section). In particular, historically, a moderate earthquake (magnitude 5.7) occurred in 1975 near Oroville due to slip on a fault in this western fault zone. Its occurrence and size provide a calibration for the intensity of the area. It should be mentioned that the peak horizontal acceleration measured in the 1975 earthquake was recorded near Oroville Dam at a value of 0.11g. Therefore, a repetition of the Oroville earthquake on lineaments 5 to 10 km to the west of Englebright Dam is not likely to be of much engineering significance, but the observation does suggest a lower bound, which is based on actual field measurements in the same tectonic environment.

Table 2

Estimated Site Motion Parameters

Source Location	Distance from Dam (km)	Predicted Maximum Magnitude (ME)	Predicted Maximum Probable Earthquake (OBE)	Horizontal Peak (Mean) Acceleration at Dam (Rock) (freq 8 Hz)	Duration at Dam (sec) (Acc 0.05g)
Cleveland Hills fault	6.5	6.25-6.5	6.0	.40g	15
Little Grass Valley fault (Melones fault zone)	25	6.0	5.5	.20g	10
Mohawk Valley fault	90	7.0	6.5	.10g	10
San Andreas fault	200	8.3	8.3	.02g	4

ME - Maximum Earthquake - defined as the severest earthquake possible at the site, on the basis of the geological and seismological evidence.

OBE - Operating Basis Earthquake - defined as being the maximum likely to occur during the life of the project.

For the present rather stringent requirements a maximum earthquake of 6.25-6.5 has been inferred for this near source (see Section IV). At what is probably the nearest source distance feasible (about 6.5 km) measured horizontal ground accelerations in such a case of about 0.40g might be expected. (Because random high frequency "spikes" of ground motion sometimes are observed on accelerograms in the near field the acceleration values in Table 2 refer to seismic waves with frequencies of 8 Hz and less.) This value is consistent with values tabulated in various studies (e.g. Schnabel and Seed, 1973; Boore et al., 1980). Because of the geological nature of the basement rock at the Englebright Dam site it is not expected that the spectrum for the ground acceleration will be unusually rich in high frequencies. The bracketed duration of this motion is likely to be about 15 sec (Bolt, 1973).

Similar correlations have been made with the other effective sources in Table 3. Each is more remote than the case treated above, so attenuation and geometrical spreading of the seismic waves is important in these cases. It can be seen that the alternative regional source (Little Grass Valley fault to the east of the dam) yields less intense shaking than the western source. Its effect is therefore included in the first case and we can disregard it in the final recommendation of predicted ground motion.

The Mohawk Valley and San Andreas fault sources are so remote that even with maximum fault rupture intensities at Englebright Dam they are clearly inconsequential compared with the near source case.

VI. Summary of Conclusions and Recommendations

The estimates given in the last section proceeded by arguing in stages from the broad regional tectonics successively back to the local conditions close to Englebright Dam. This procedure led to the establishment of design ground motion parameters that can be adopted with considerable confidence. The reason is that, in this case, a moderate earthquake cannot be ruled out for a fault source from 6 to 10 km away from Englebright Dam. As it turns out, more remote sources with some reasonable likelihood of activity within 100 km of the project would generate less severe ground shaking at the project (see Table 2).

What has emerged from the assessment, therefore, is a single design maximum earthquake. The earthquake is taken to be generated by sudden slip (predominantly normal faulting) along a southern extension of what has been mapped as the Cleveland Hill fault (or a related fault, perhaps part of the Swain Ravine lineament) to the west of the reservoir. The magnitude of this maximum earthquake is about M_L = 6.25 to 6.5. It must be stressed, however, that the geological and seismological evidence indicates that an earthquake of this magnitude is very unlikely within the lifetime of the dam and its adoption would provide an upper bound to feasible ground motions for engineering analysis. In my assessment, the maximum probable magnitude for an earthquake centered as close as 10 km from the dam in the next 50 to 100 years is no more than about 6.0. Other active faults with significantly more frequent slip are located to the west at a distance of over 90 km; seismic wave damping over such distances ensures that the ground motions at Englebright Dam for even the largest earthquakes on these presently active faults would not exceed about 0.1g.

Recommendations for site motion parameters associated with the maximum earthquake on the Cleveland Hill fault near the reservoir are as follows: For frequencies of 8 Hz or less, a mean peak horizontal acceleration of 0.40g and a bracketed duration of 20 sec. It is further recommended that the time history should contain a longer period pulse of the type observed on accelerograms near to rupturing faults. This pulse is associated with the fling of the fault rebound. The ground motion velocity associated with this pulse should be about 30 cm/sec (see Boore et al., 1980).

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FIGURE CAPTIONS

Figure 1. Regional map showing Englebright Dam and generalized faults and lineaments in the region of Englebright Dam (after Woodward-Clyde, 1976). Position of reservoir shown by arrow.

Figure 2. Map of relevant fault zones and epicenters of known earthquakes, magnitude 3.0 or greater, within 50 km of dam site. (See Appendix A for detailed list.) (Map taken from Woodward-Clyde Consultants, 1976 - for P.G.&E.) Position of reservoir shown as large full circle.

APPENDIX 1

Seismicity within 100 km of the site (6/6/1934 to 3/31/82). Magnitude greater or equal to 3.5. Hypocenters are computed using readings at Seismographic Stations of the University of California, Berkeley and selected other stations as appropriate. The list is in the form of a computer print out from a master file kept on magnetic tape at the Seismographic Stations, University of California, Berkeley.

APPENDIX B

SELECTION OF ACCELEROGRAMS FOR SEISMIC SAFETY EVALUATION

Selection of Accelerograms for Seismic Safety Evaluation of Englebright Dam, California

by

H. Bolton Seed

The Englebright Dam, completed in 1941, is located on the Yuba River about 25 miles northeast of Marysville, California. The dam is a concrete arch structure with a maximum height of 260 ft and a length of about 1,140 ft.

A report on the seismicity of the area in which the dam is located entitled "Seismicity and Seismic Intensity Study, Englebright Dam, California" has been prepared for the Sacramento District, Corps of Engineers by Dr. Bruce A. Bolt. This report concludes that the maximum earthquake for which the safety of the dam should be evaluated is a Magnitude M_L = 6-1/4 to 6-1/2 event occurring at a distance of 6 to 10 kms from the dam along a southern extension of what has been mapped as the Cleveland Hill fault (or a related fault, perhaps part of the Swain Ravine lineament). The report stresses however "that the geological and seismological evidence indicates that an earthquake of this magnitude is very unlikely within the lifetime of the dam and its adoption would provide an upper bound to feasible ground motions for engineering analysis."

Based on this recommendation, two accelerograms representative of this type of event have been developed for use in engineering studies of the dam. These accelerograms are designated:

M6.5 - 8K - 83 This accelerogram is representative of the 84-percentile level of ground motions which could be expected to occur at a rock outcrop as a result of a Magnitude 6-1/2

earthquake occurring 6 to 10 kms from the site. It has the following characteristics:

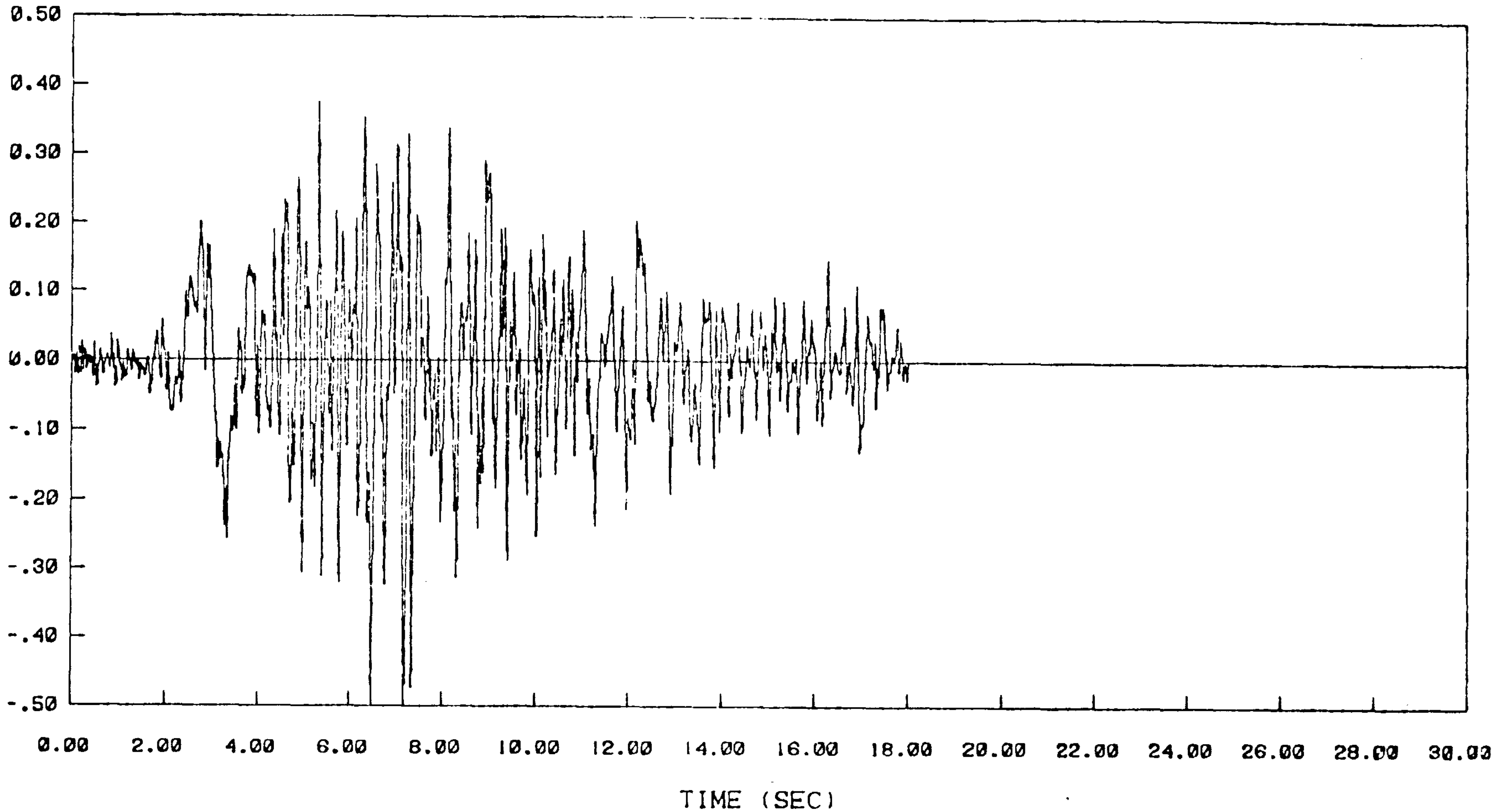
Peak acceleration	= 0.5g
Peak velocity	= 37.5 cm/sec
Duration	≃ 18 sec

M6.4 - 8K - 83 This accelerogram is representative of the 84-percentile level of ground motions which could be expected to occur at a rock outcrop as a result of a Magnitude 6.4 earthquake occurring 6 to 10 kms from the site. It has the following characteristics:

Peak acceleration	= 0.5g
Peak velocity	= 41.5 cm/sec
Duration	≃ 16 sec

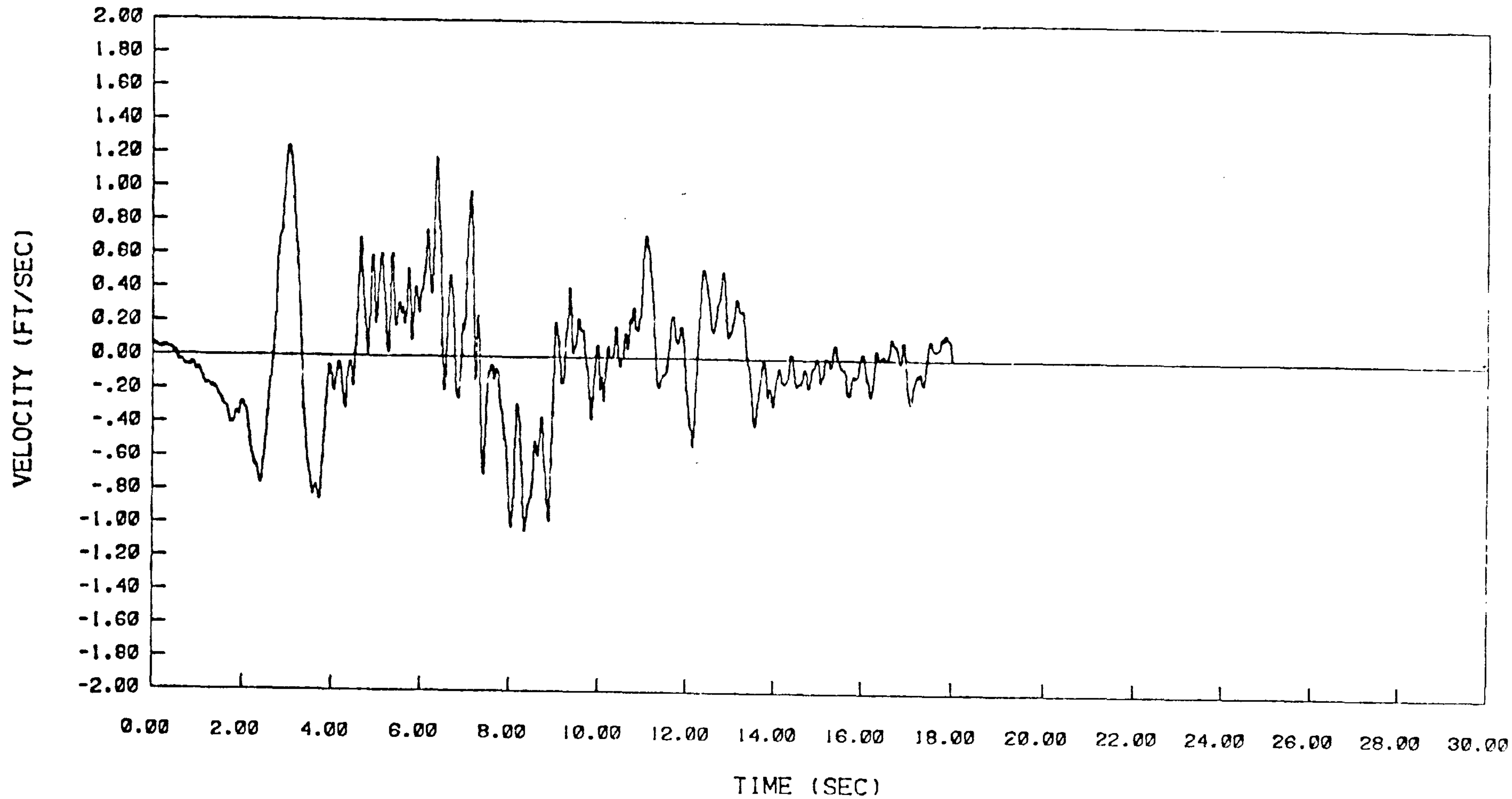
Plots of acceleration and velocity as a function of time for these accelerograms are attached, together with acceleration response spectra for the motions plotted for a damping ratio of 0.05.

Card decks for the motions are being sent to the office of the Sacramento District, Corps of Engineers, by separate mail.



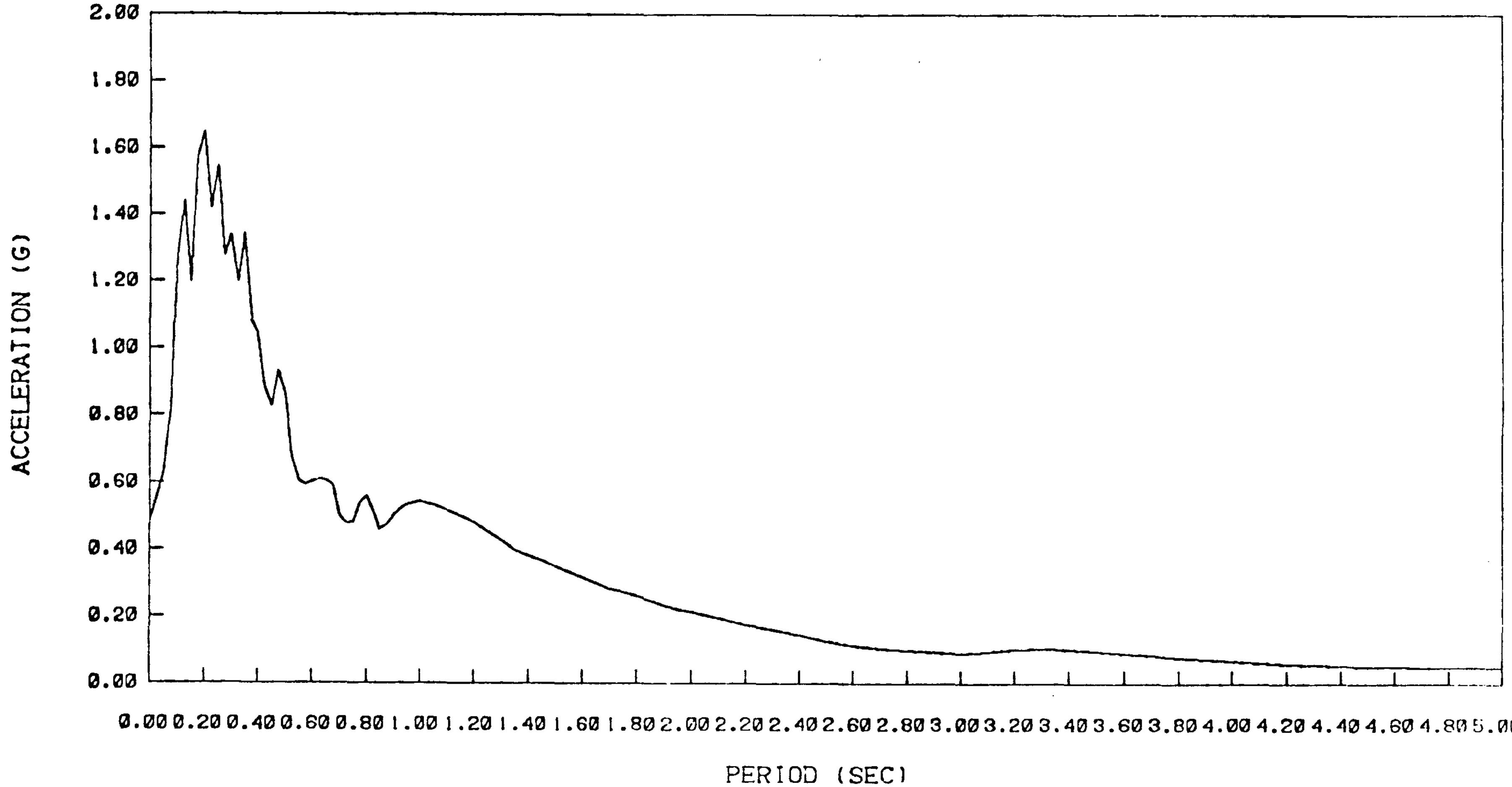
ACCELERATION TIME HISTORY

M6.5 - 8K - 83



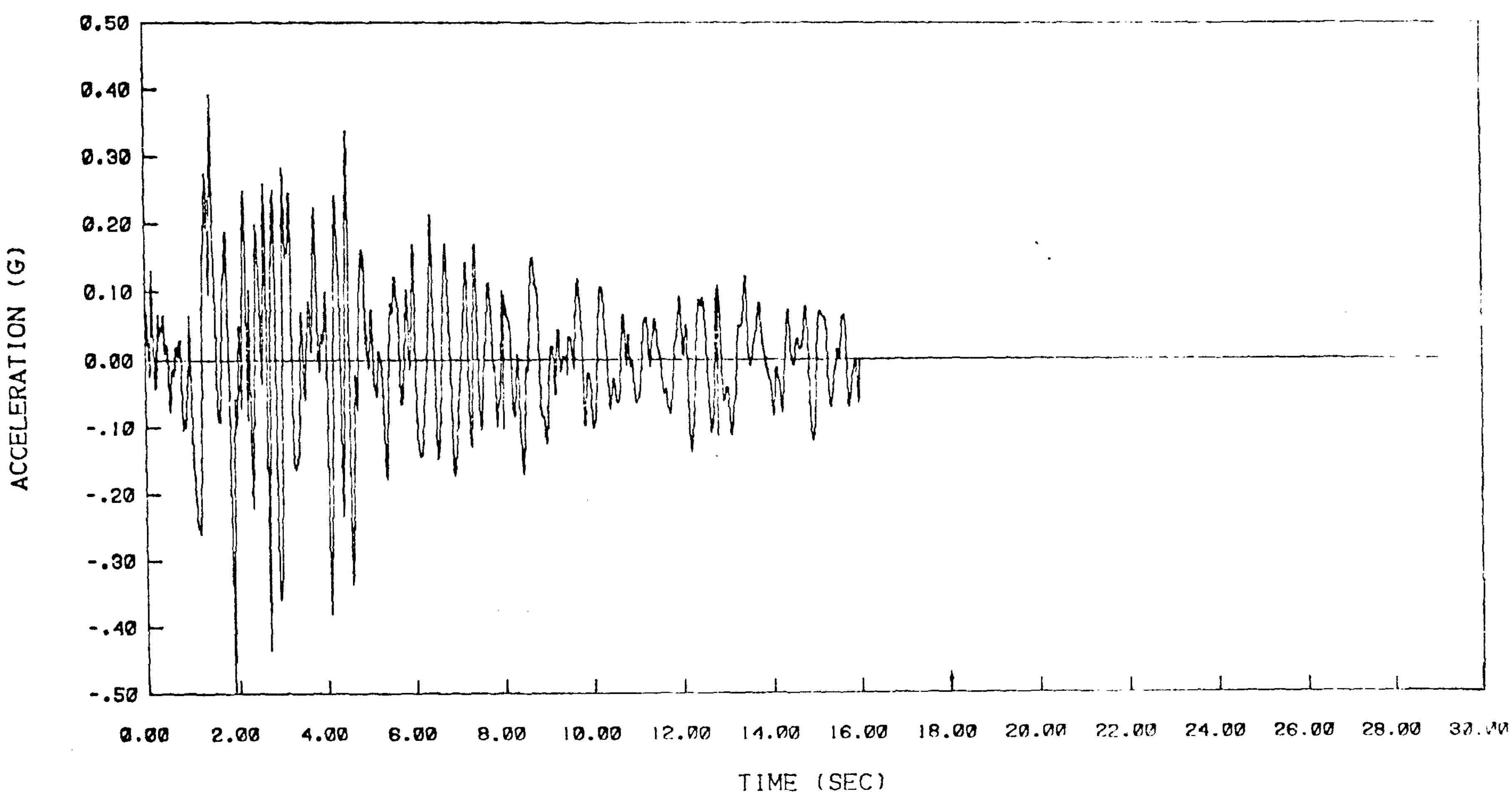
VELOCITY TIME HISTORY

M6.5 - 8K - 83



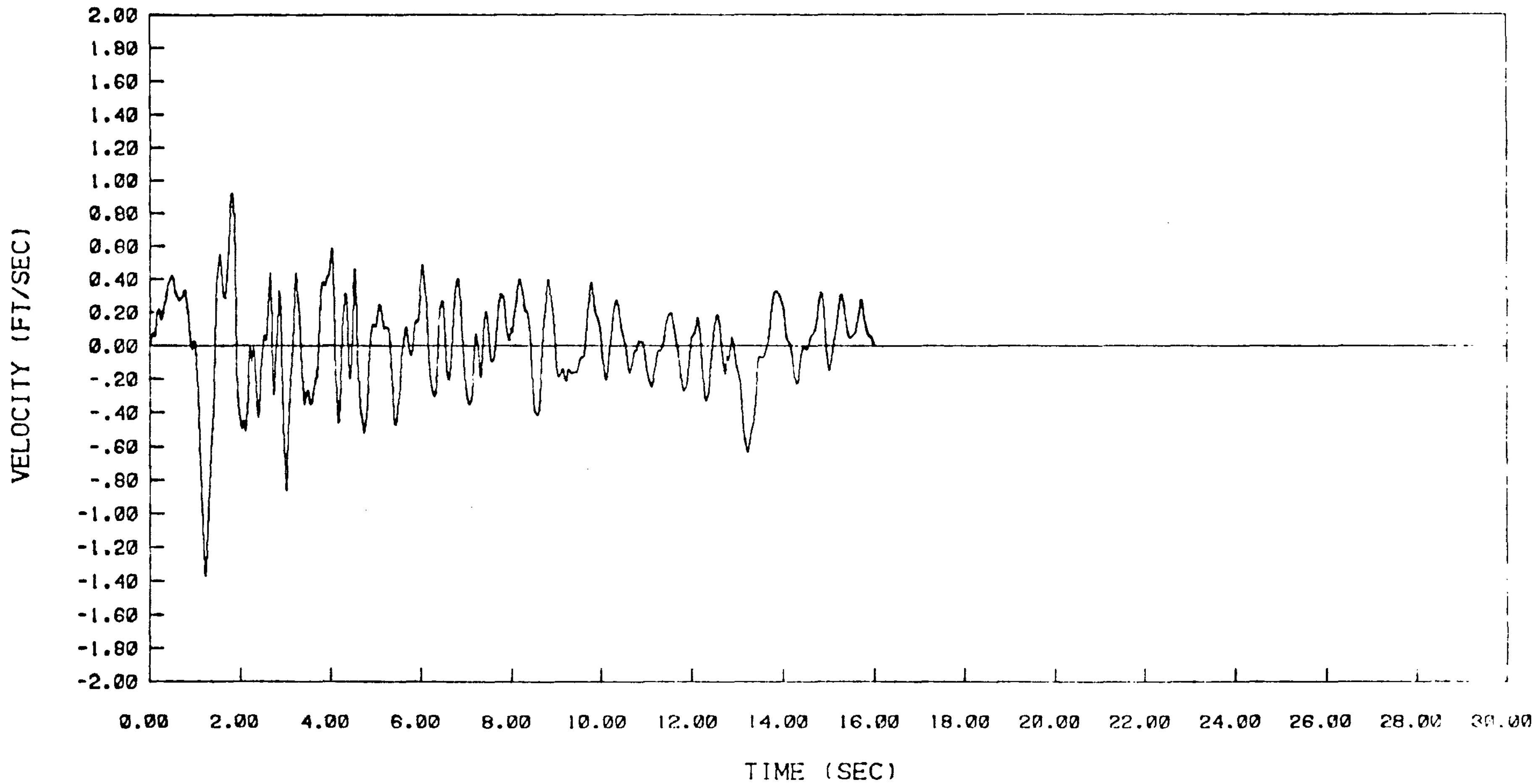
PLOT OF ACCELERATION RESPONSE SPECTRUM

M6.5 - 8K - 83



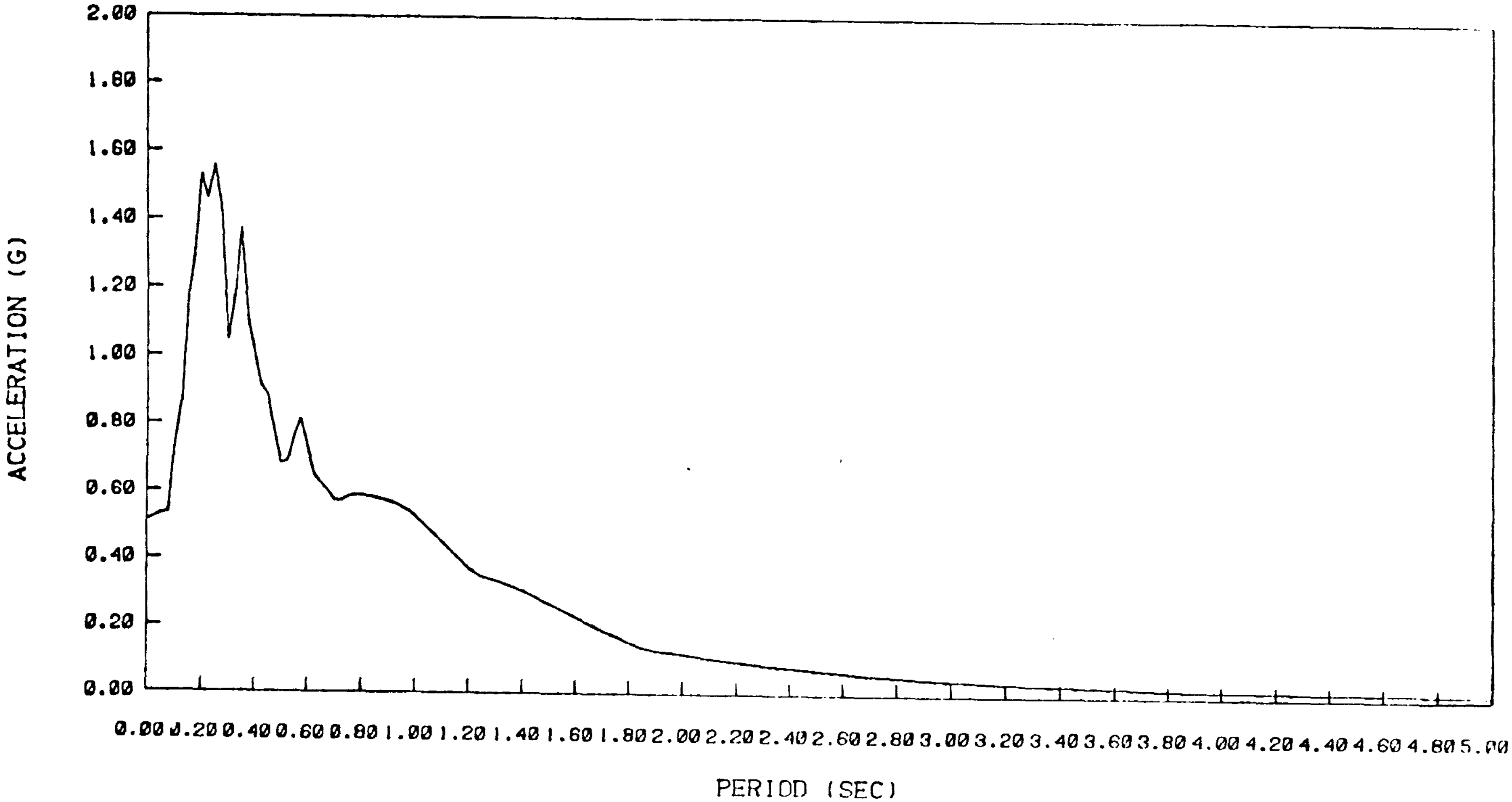
ACCELERATION TIME HISTORY

M6.4 - 8K - 83



VELOCITY TIME HISTORY

M6.4 - 8K - 83



PLOT OF ACCELERATION RESPONSE SPECTRUM

M6.4 - 8K - 83

APPENDIX C

JULY 12, 1985 LETTER BY DR. BOLT AND DR. SEED

H. Bolton Seed, Inc.
623 CROSSRIDGE TERRACE. ORINDA. CALIFORNIA 94563
(415) 254-3036

July 12, 1985

Sacramento District, Corps of Engineers
Engineering Division, Civil Design Branch
650 Capital Mall
Sacramento, CA 95814-4794

Attention: Mrs. Susan Kristoff

In accordance with the requirements of Purchase Orders Nos. DACW05-85-P-2200 and DACW08-85-P-2194 the undersigned have reviewed the conditions at the site of the proposed Englebright Dam for the purpose of determining:

1. Appropriate accelerograms for use in the analysis of the dam for motions normal to the axis of the dam, parallel to the axis of the dam and in the vertical direction;

and 2. Recommendations concerning the possible effects of topography on the characteristics of the design motions which should be considered to act at the base of the dam.

In a previous report, Professor Bolt had recommended that an appropriate characterization of the safety evaluation earthquake for this project would be a Magnitude $M_L = 6\frac{1}{4}$ to $6\frac{1}{2}$ event occurring at a distance of about 8 kms from the site, recognizing that "the geologic and seismologic evidence indicates that an earthquake with this magnitude is very unlikely within the lifetime of the dam and its adoption would provide an upper bound to feasible ground motions for engineering analysis".

Based on this recommendation, Professor Seed recommended two design accelerograms, both intended to represent the 84th percentile level of ground motions from a magnitude $6\frac{1}{2}$ event occurring at a distance of 8 kms.

These motions were designated:

M 6.5 - 8K - 83

and M 6.4 - 8K - 83

respectively. Both accelerograms had a peak ground acceleration of 0.5g, a peak velocity in the range 37 to 42 cm/sec and a duration of shaking of about 16 seconds.

They were intended to provide an appropriately conservative description of the free-field motions which the dam should be designed to withstand for earthquake shaking in a direction normal to the axis of the dam.

Since ground motions during any earthquake are more appropriately described by three components of earthquake shaking and such motions can be accommodated in analyses of structural response, it is now desired to extend the previous work to include recommendations for motions developed simultaneously in three directions:

(1) normal to the axis of the dam

(2) along the axis of the dam

(3) vertical motions.

Studies of earthquake motions conducted for the purpose of evaluating the simultaneous effects of three components of motion have been described by Shoja-Taheri and Bolt (1977). Based on the results of these studies and the writers' knowledge of the characteristics of earthquake ground motions, we would recommend the use of the following accelerograms for evaluating the effects of multi-directional shaking in the analysis of Englebright Dam:

1. Analyses using the motion M6.5 - 8K - 83 in a direction normal to the axis of the dam

In conjunction with this accelerogram for motions developed in a direction normal to the axis we would propose that the two complementary

components of motion be determined as follows:

(a) The accelerogram for motions parallel to the axis should be taken to be the same as that for the motions normal to the axis but with a phase shift in the motions corresponding to a time interval of 0.45 seconds; i.e. the accelerogram would be the same as M6.5 - 8K - 83 but it would be considered to start in a direction parallel to the axis 0.45 seconds later than the start of the accelerogram for motions acting in a direction normal to the axis.

(b) The accelerogram for vertical motions would be the same as the M6.4 - 8K - 83 accelerogram but the frequency of the motions would be increased by a factor of 1.5 and the amplitudes would be reduced by a scaling factor of 0.5.

2. <u>Analyses using the motion M6.4 - 8K - 83 in a direction normal to the axis of the dam</u>

In analyses using this accelerogram for motions developed in a direction normal to the axis, we propose that the complementary components of motion be determined as follows:

(a) The accelerogram for motions parallel to the axis should be taken to be the same as the motions normal to the axis but with a phase shift in the motions corresponding to a time interval of 0.5 seconds, i.e. the accelerogram would be the same as M6.4 - 8K - 83 but it would be considered to start in a direction parallel to the axis 0.5 seconds later than the start of the accelerogram for motions in a direction normal to the axis.

(b) The accelerogram for vertical motions would be the same as the M6.4 - 8K - 83 accelerogram but the frequency of the motions would be increased by a factor of 1.5 and the amplitudes would be reduced by a scaling factor of 0.5.

With regard to the possibility of topographic effects on the amplitudes of rock motions in the abutments, we do not consider that such effects are likely to be significant for the topographic changes associated with the abutments of this dam. Accordingly we recommend that the same time histories of rock motions be applied to all points of the base. However it might be desirable to consider the possibility of travelling wave effects along the base of the dam in performing the dynamic response analyses. In this case we would suggest that differential motions along the base should correspond to an effective velocity of wave motions of about 10,000 fps.

Finally we note that the accelerograms described above are intended to describe the free-field motions for the Safety Evaluation Earthquake for this site and these are not necessarily the same as the design earthquake motions.

We trust these recommendations are responsive to your needs. We will be pleased to meet with you for further discussion if you so desire.

Sincerely yours,

Bruce A. Bolt

H. Bolton Seed

H. Bolton Seed

APPENDIX D

CONCRETE AND FOUNDATION ROCK TEST DATA

TABLE D-1: ENGLEBRIGHT DAM STATIC LOAD TEST DATA
FROM THE CORPS OF ENGINEERS

Sample Number	Compressive Strength, psi	Tensile Strength, psi	Modulus of Elasticity, x1000 ksi	Poisson's Ratio
1C-A	6600	-	5.28	.150
B	5800	510	5.30	.137
C	6350	652	4.89	.145
D	5880	526	4.14	.106
E	5760	558	3.69	.120
F	6310	-	4.16	.134
G	8650	540	5.32	.154
L	7760	665	4.60	.148
M	-	489	-	-
N	7570	603	4.85	.130
P	4780	-	3.36	.193
Q	6510	-	4.01	.135
W	5220	721	6.68	.192
Z	-	628	-	-
CC	7710	679	5.42	.132

TABLE D-2: ENGLEBRIGHT DAM RAPID COMPRESSION TEST DATA FROM U.S. BUREAU OF RECLAMATION

Specimen identification	Diameter (in)	Length (in)	Length / diameter	Time (ms)	Load (lb)	Cross-sectional area (in^2)	Compressive strength (lb/in^2)
IC-A1	5.88	12.03	2.05	75.0	195,725	27.15	7,208
IC-BB1	5.90	11.90	2.02	67.2	222,284	27.34	8,130
IC-F	5.89	11.78	2.00	68.4	131,164	27.25	4,814
IC-G1	5.91	11.80	2.00	68.5	171,617	27.43	6,256
IC-G2	5.91	11.36	1.92	78.0	245,984	27.43	*8,931
IC-I1	5.91	12.15	2.06	71.0	183,058	27.43	6,673
IC-I2	5.91	12.04	2.04	80.2	190,843	27.43	6,957
IC-J1	5.90	11.86	2.01	62.2	152,820	27.34	5,590
IC-J2	5.89	11.94	2.03	64.5	172,862	27.25	6,344
IC-K	5.90	12.08	2.05	80.1	212,093	27.34	7,758
IC-M1	5.88	12.00	2.04	70.5	164,670	27.15	6,064
IC-O	5.89	12.08	2.05	60.3	152,430	27.25	5,594
IC-R1	5.89	12.11	2.06	55.3	146,691	27.25	5,384
IC-S1	5.88	11.92	2.03	62.9	194,521	27.15	7,163
IC-U1	5.87	12.00	2.04	74.7	198,585	27.06	7,338
IC-V1	5.91	11.82	2.00	62.1	158,541	27.43	5,779
IC-V2	5.90	12.06	2.04	76.7	164,262	27.34	6,008
IC-X1	5.92	12.16	2.05	56.3	188,778	27.53	6,858
IC-Z1	5.91	11.87	2.01	68.5	213,295	27.43	7,775
Average							6,664

* Corrected for L/d <2.00.

TABLE D-3: ENGLEBRIGHT DAM RAPID SPLITTING TENSION TEST DATA FROM U.S. BUREAU OF RECLAMATION

Specimen identification	Diameter (in)	Length (in)	Time (ms)	Load (lb)	Tensile strength (lb/in^2)
IC-AA1	5.89	11.91	72.7	62,144	564
IC-BB2	5.90	12.07	72.5	78,090	698
IC-DD1	5.89	12.00	74.0	64,768	583
IC-DD2	5.89	12.07	50.9	49,630	444
IC-G	5.90	12.15	54.0	91,613	814
IC-H1	5.88	11.81	57.1	62,346	572
IC-H2	5.85	11.75	59.9	59,318	549
IC-K	5.89	12.22	77.3	58,713	519
IC-M2	5.89	11.94	61.4	59,924	542
IC-R2	5.89	12.27	45.1	62,951	554
IC-S2	5.89	12.18	69.2	72,438	643
IC-T	5.90	12.00	81.9	67,998	611
IC-U2	5.88	11.59	84.5	62,548	584
IC-Y	5.89	11.89	56.3	68,199	620
IC-Z2	5.89	10.97	65.2	49,630	489
Average					586

TABLE D-4: ENGLEBRIGHT DAM CHORD MODULUS OF ELASTICITY AND POISSON'S RATIO DATA FROM U.S. BUREAU OF RECLAMATION

Specimen identification	Stress at 40% ultimate stress S_2 (lb/in^2)	Stress at longitudinal strain $\epsilon_1 = 0.000050$ S_1 (lb/in^2)	Longitudinal strain at 40% ultimate stress ϵ_2 (μ in/in)	Chord modulus of elasticity E_c (lb/in^2)	Transverse strain at 40% ultimate stress ϵ_{t2} (μ in/in)	Transverse strain at longitudinal strain $\epsilon_1 = 0.000050$ ϵ_{t1} (μ in/in)	Poisson's ratio r
IC-A1	2883	207	631	4,605,852	107	0	0.184
IC-BB1	3252	272	644	5,016,835	93	1	0.155
IC-F	1926	192	523	3,665,962	53	0	0.112
IC-G1	2502	249	502	4,984,513	151	30	0.268
IC-G2	3572	248	783	4,534,789	139	0	0.190
IC-I1	2669	254	552	4,810,757	122	10	0.223
IC-I2	2783	191	714	3,903,614	103	0	0.155
IC-J1	2236	220	484	4,645,161	33	0	0.076
IC-J2	2538	162	533	4,919,255	115	0	0.238
IC-K	3103	175	769	4,072,323	156	0	0.217
IC-M1	2426	198	598	4,065,693	122	0	0.223
IC-O	2238	206	524	4,286,920	93	0	0.196
IC-R1	2154	246	506	4,184,211	114	2	0.246
IC-S1	2865	237	549	5,266,533	100	5	0.190
IC-U1	2935	234	624	4,705,575	98	0	0.171
IC-V1	2312	249	412	5,698,895	129	2	0.351
IC-V2	2403	246	617	3,804,233	152	6	0.257
IC-X1	2743	245	525	5,258,947	113	1	0.236
IC-Z1	3110	244	561	5,608,611	111	0	0.217
Average				4,633,615			0.206

$$E_c = \frac{(S_2 - S_1)}{(\epsilon_2 - 0.000050)} \qquad r = \frac{(\epsilon_{t2} - \epsilon_{t1})}{(\epsilon_2 - 0.000050)}$$

TABLE D-5: COMPARISON BETWEEN STATIC LOAD TEST DATA (COE) AND RAPID LOAD TEST DATA (USBR) OF ENGLEBRIGHT DAM

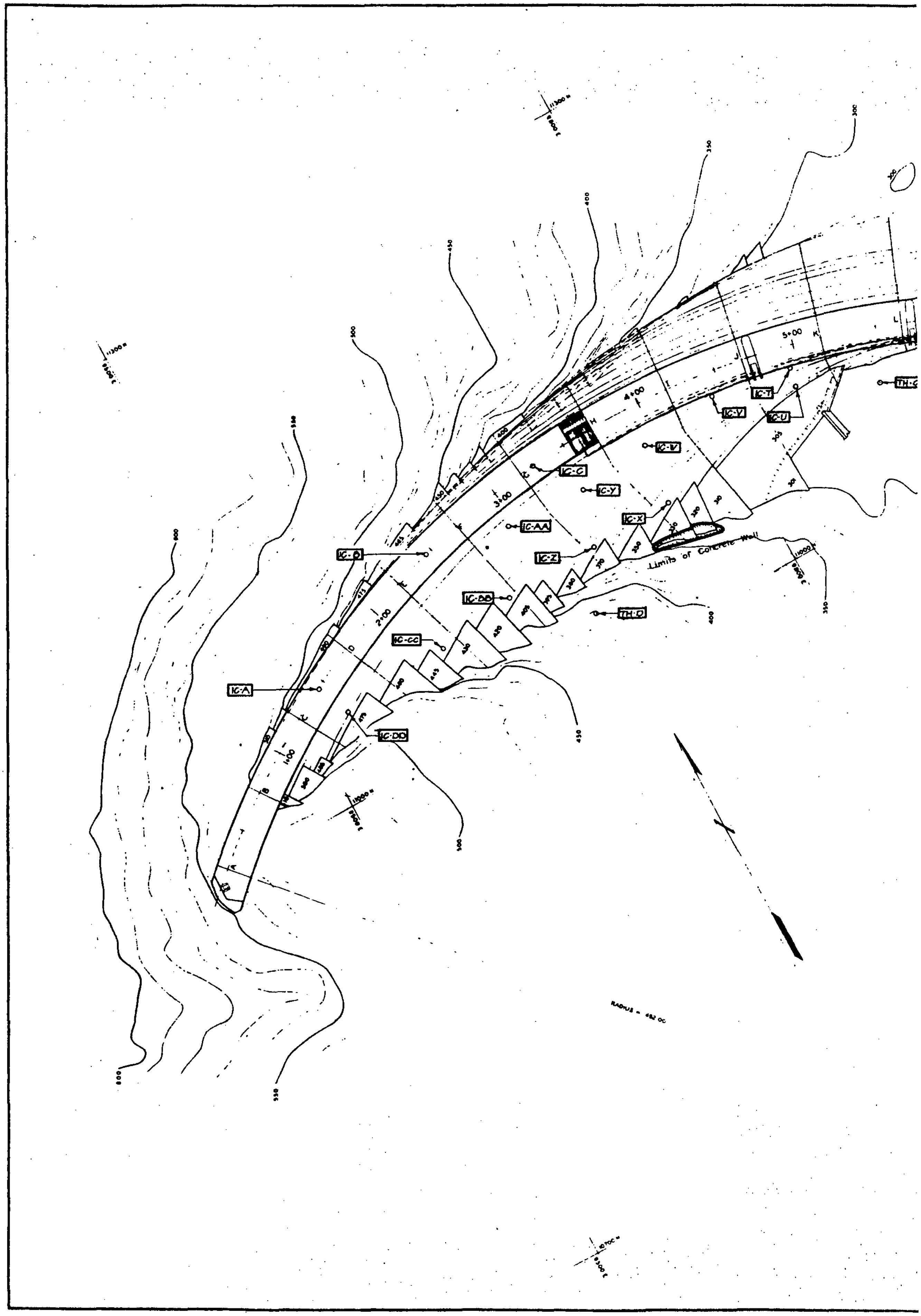
PARAMETER TEST	STATIC LOAD TESTS (SPD LAB)			RAPID LOAD TESTS (USBR LAB)		
	MEAN	80% EXCEEDANCE	STANDARD DEVIATION	MEAN	80% EXCEEDANCE	STANDARD DEVIATION
COMPRESSIVE STRENGTH	6530 psi	5600 psi	1110 psi	6660 psi	5770 psi	1060 psi
SPLITTING TENSILE STRENGTH	597 psi	532 psi	77.3 psi	585 psi	509 psi	90 psi
CHORD MODULUS OF ELASTICITY	4.75×10^6 psi		0.888×10^6 psi	4.63×10^6 psi		0.596×10^6 psi
POISSON'S RATIO	0.14		.025 psi	0.21		0.06

TABLE D-6: SUMMARY OF ENGLEBRIGHT DAM NX ROCK CORE TESTING

NO.	HOLE	DEPTH (FT)	LOAD RATE (PSI/S)	UNCONFINED COMPRESSIVE STRENGTH, σ_1 (PSI)	MODULUS OF ELASTICITY, E* ($X10^6$ PSI)	POISSON'S RATIO, ν
1	A	9.7-10.6	117.4	48,130	14.2	.22
2	A	23.4-24.2	57.4	13,480	12.0 (1)	.21 (1)
3	A	47.6-49.2	24.4	21,600	10.5 (2)	.22 (2)
4	B	8.5-10.7	24.4	20,910	11.1 (3)	.30 (3)
5	B	20.7-21.4	51.4	16,460	11.0 (4)	.14 (4)
6	B	48.2-48.9	31.4	28,720	11.8	.27
7	C	10.0-10.8	41.2	41,810	12.9	.26
8	C	19.2-20.3	57.1	27,690	13.4	.29
9	C	31.0-31.8	60.5	14,530	11.7 (5)	.36 (5)
10	D	8.4-9.0	43.1	31,480	13.6	.20
11	D	22.5-25.0	38.6	64,480	14.0	.24
12	D	41.3-42.4	36.9	47,200	13.6	.19

* Taken at 40% of unconfined compressive strength unless otherwise noted

(1) 17% of unconfined compressive strength
(2) 14% " " " "
(3) 13% " " " "
(4) 9% " " " "
(5) 29% " " " "



IC-A
IC-B
IC-C
IC-T
IC-U
IC-V
IC-W
IC-X
IC-Y
IC-Z
IC-AA
IC-BB
IC-CC
IC-DD
TH-D
Limits of Concrete Wall
1+00
2+00
3+00
4+00
5+00

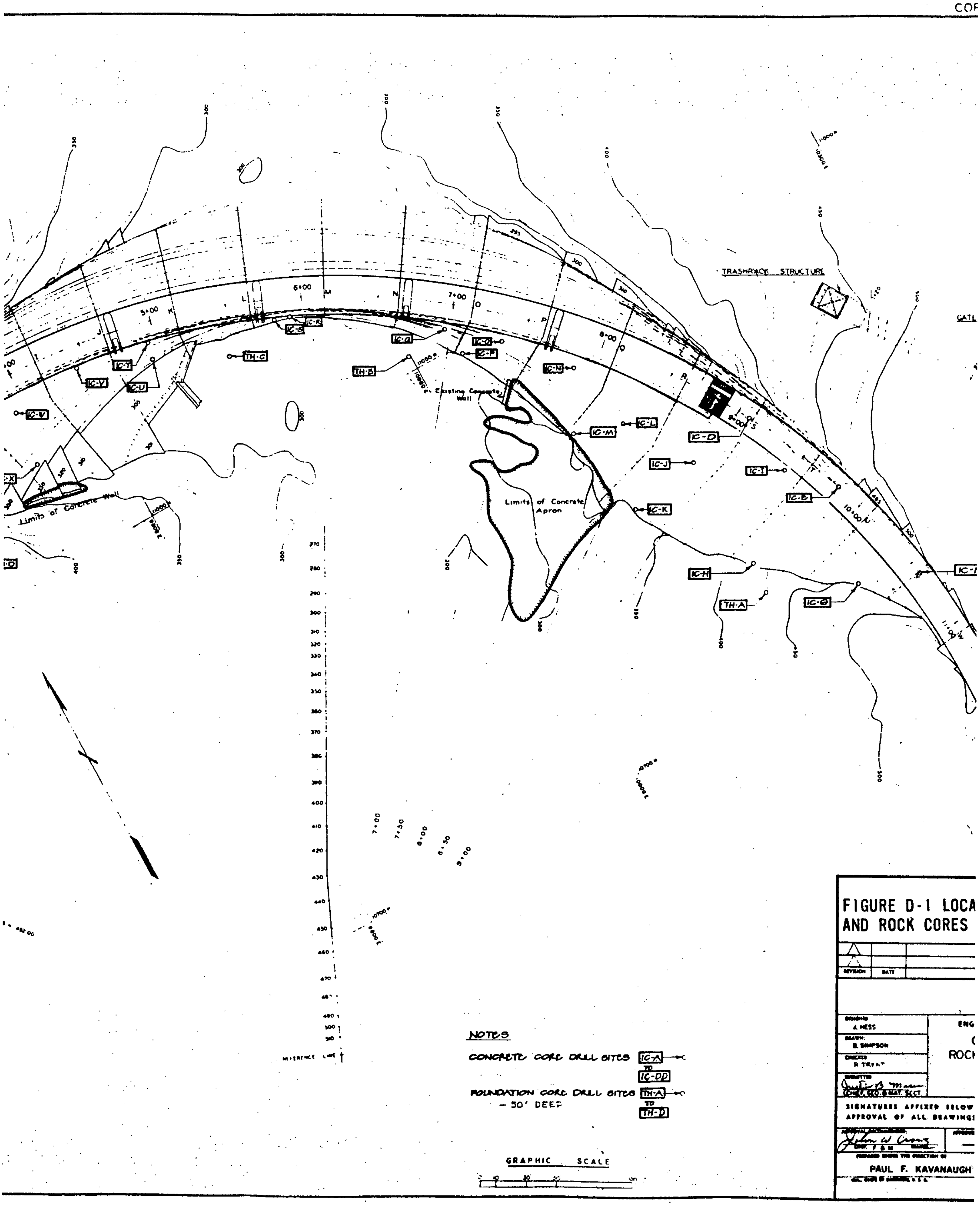


FIGURE D-1 LOCA
AND ROCK CORES

CORPS OF ENGINEERS, U.S. ARMY

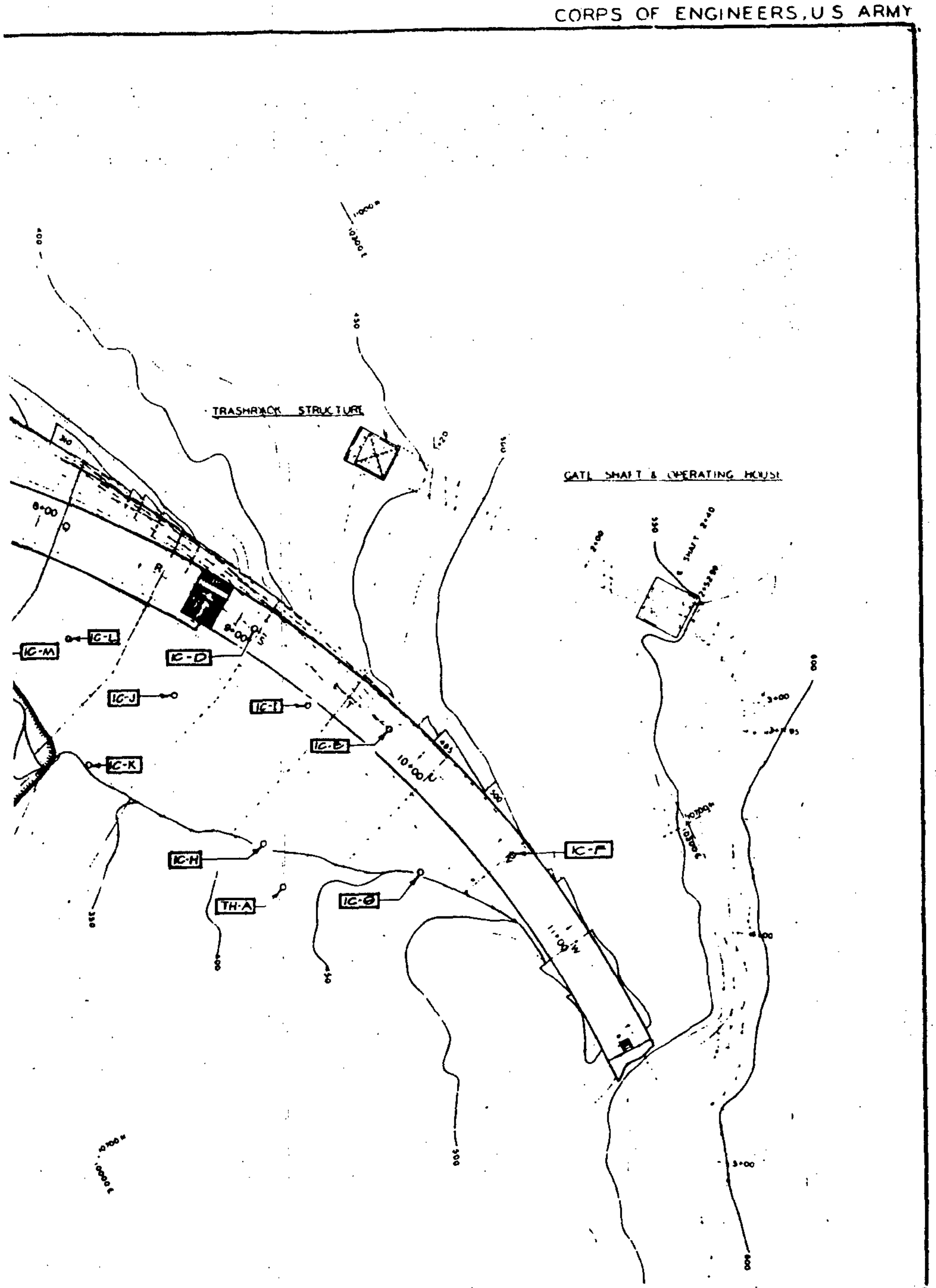


FIGURE D-1 LOCATIONS OF CONCRETE CORES AND ROCK CORES - PLAN VIEW

REVISION	DATE	DESCRIPTION	BY	BY

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

DESIGNED: J. HESS
DRAWN: B. SIMPSON
CHECKED: R. TREAT
SUBMITTED: CHIEF, GEO. & MAT. SECT.

ENGLEBRIGHT AND PINE FLAT DAMS, CALIFORNIA

CONCRETE CORE DRILLING,
ROCK CORE DRILLING AND TESTING

ENGLEBRIGHT DAM

PLAN VIEW

SIGNATURES AFFIXED BELOW INDICATE OFFICIAL RECOMMENDATION AND APPROVAL OF ALL DRAWINGS IN THIS SET AS INDEXED ON THIS SHEET

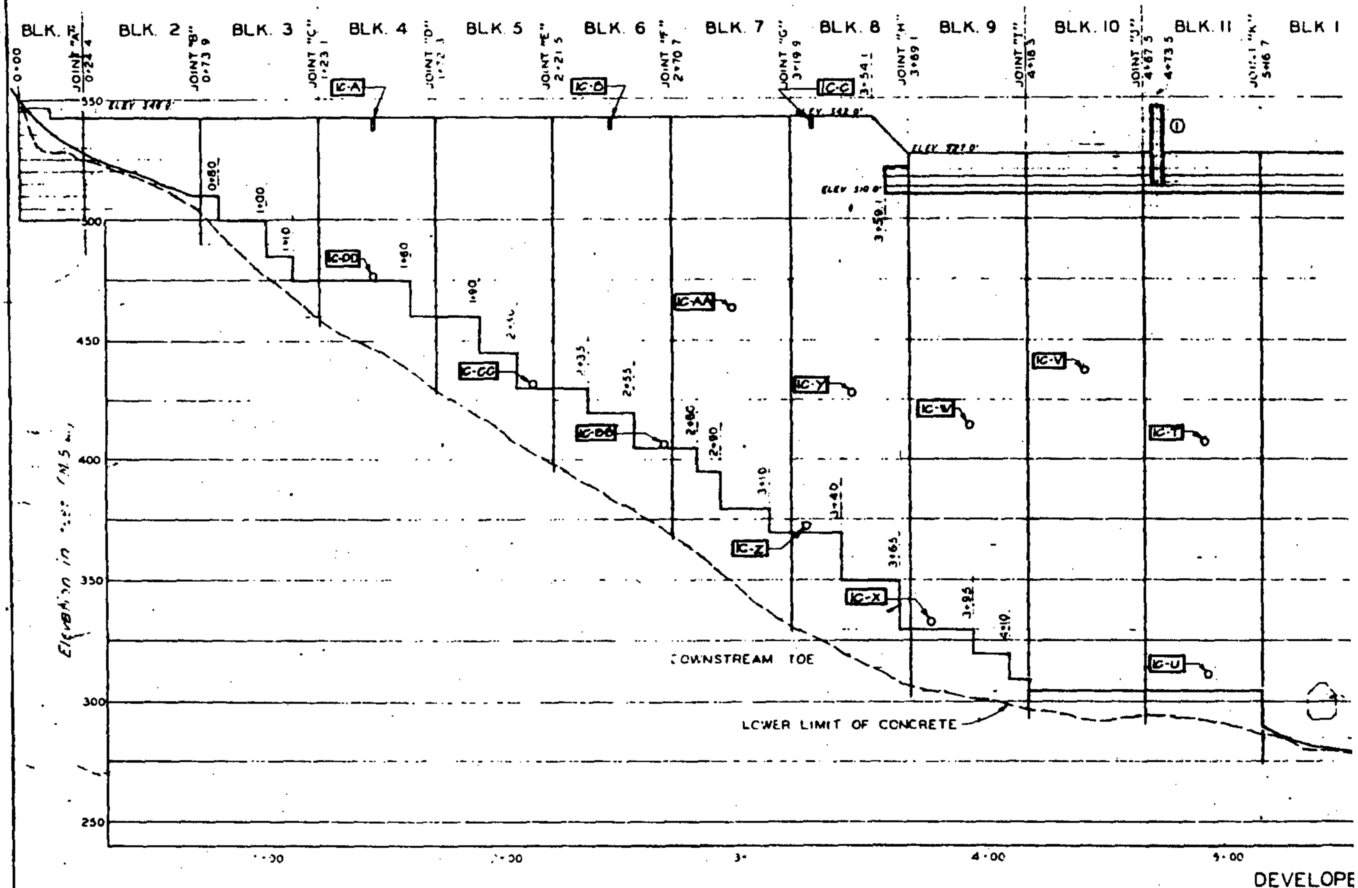
APPROVAL RECOMMENDED: | APPROVED | DATE

PREPARED UNDER THE DIRECTION OF

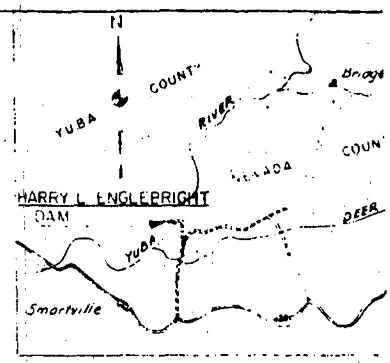
PAUL F. KAVANAUGH

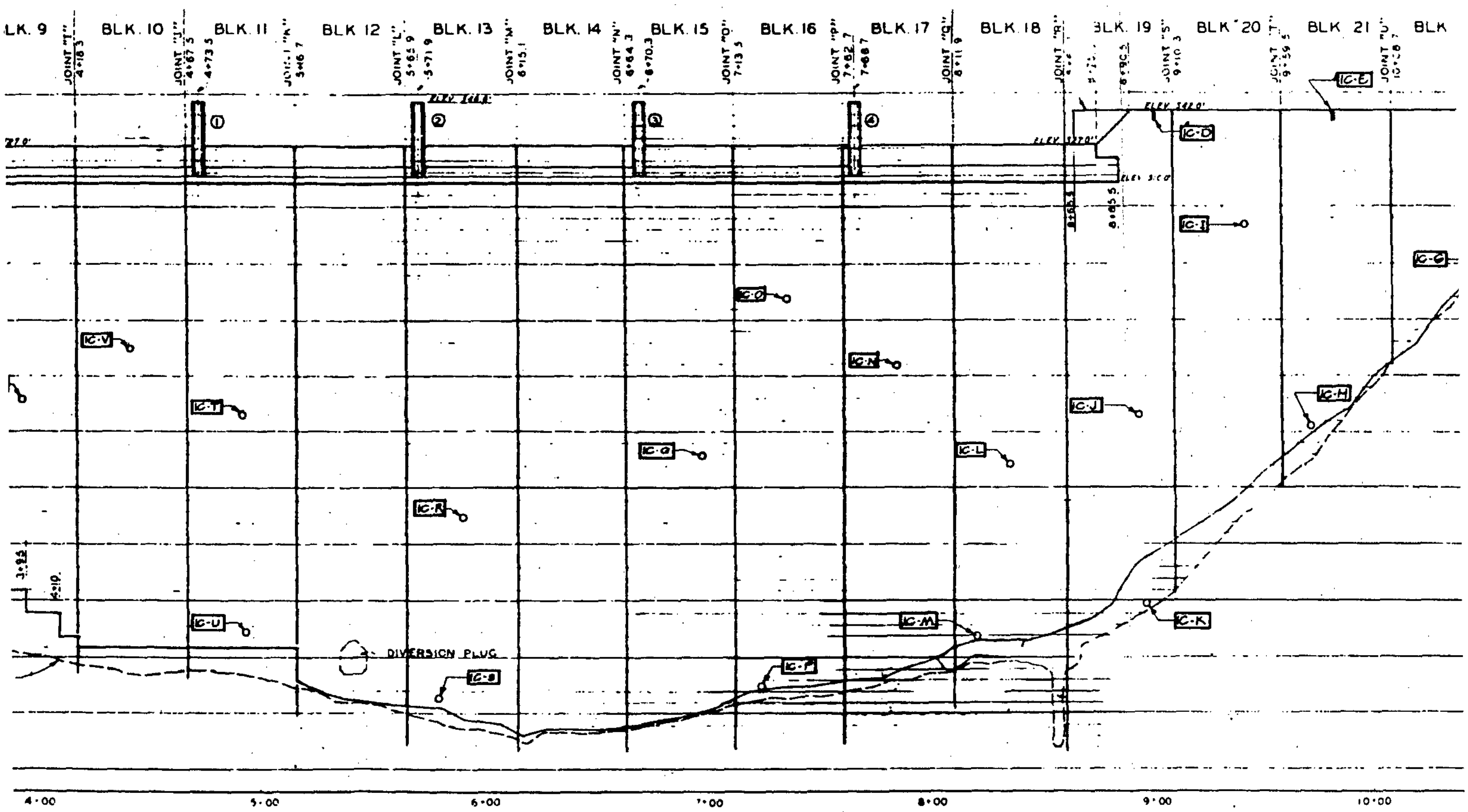
SCALE: 1" = 30'-0" | SPEC. No.

SHEET 1 | FILE No. 8-1-844



BLK. 1
BLK. 2
BLK. 3
BLK. 4
BLK. 5
BLK. 6
BLK. 7
BLK. 8
BLK. 9
BLK. 10
BLK. 11
JOINT "A"
JOINT "B"
JOINT "C"
JOINT "D"
JOINT "E"
JOINT "F"
JOINT "G"
JOINT "H"
JOINT "J"
JOINT "K"
IC-A
IC-B
IC-C
IC-DD
IC-AA
IC-CC
IC-BB
IC-Y
IC-W
IC-V
IC-T
IC-Z
IC-X
IC-U
DOWNSTREAM TOE
LOWER LIMIT OF CONCRETE
Elevation in feet (MSL)
550
500
450
400
350
300
250
4+00
5+00
DEVELOPE





DEVELOPED PROFILE LOOKING UPSTREAM

SCALE 1" = 30'-0"

GRAPHIC

NOTE:

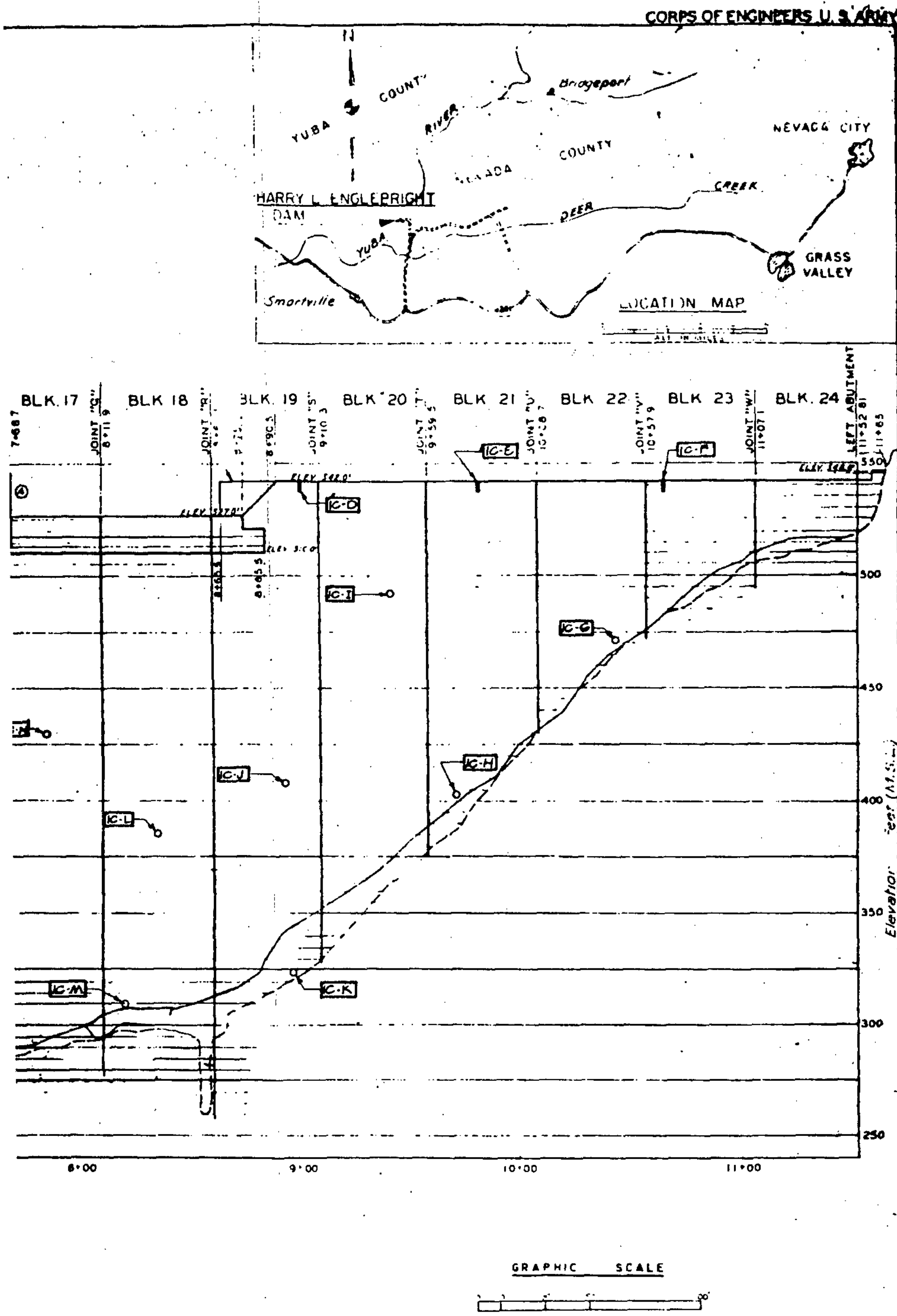
1. 50' DEEP FOUNDATION HOLES TH-A THROUGH TH-D SHOWN ON SHEET 1
2. CONCRETE CORE DRILL SITES IC-A TO IC-DD

FIGURE D-2 LOCAT
-ELEVATION VIEW

REVISION	DATE

DESIGNED: J. HESS
DRAWN: B. SIMPSON
CHECKED: K. TREAT
SUBMITTED: Justin B. Moses

ENGLEB
CON
ROCK C
E



ES [TH-A]
SHEET 1

TES [IC-A] TO [IC-DD]

FIGURE D-2 LOCATIONS OF CONCRETE CORES -ELEVATION VIEW

REVISION	DATE	DESCRIPTION	BY	BY

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

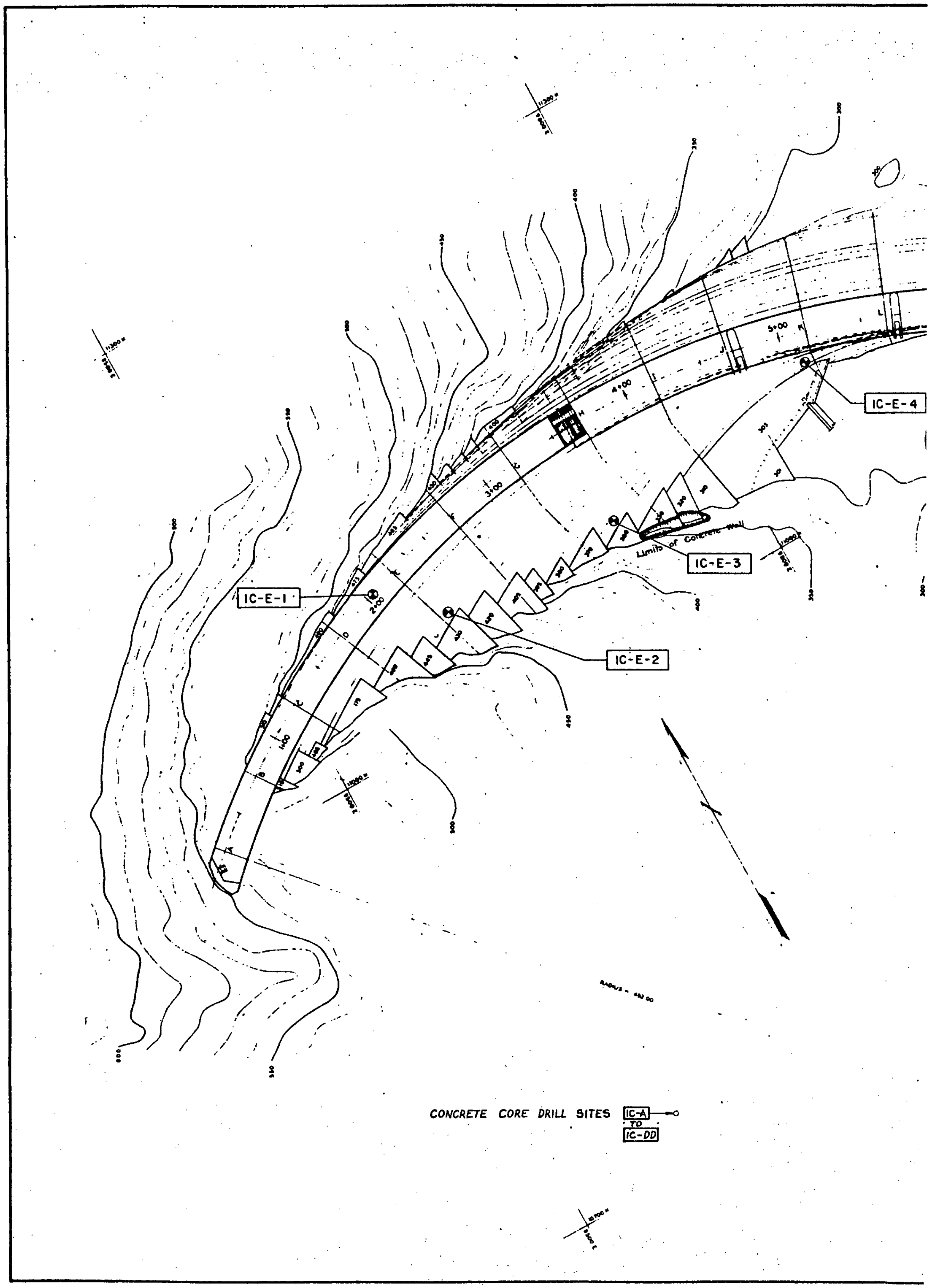
ENGLEBRIGHT AND PINE FLAT DAMS, CALIFORNIA

CONCRETE CORE DRILLING,
ROCK CORE DRILLING AND TESTING

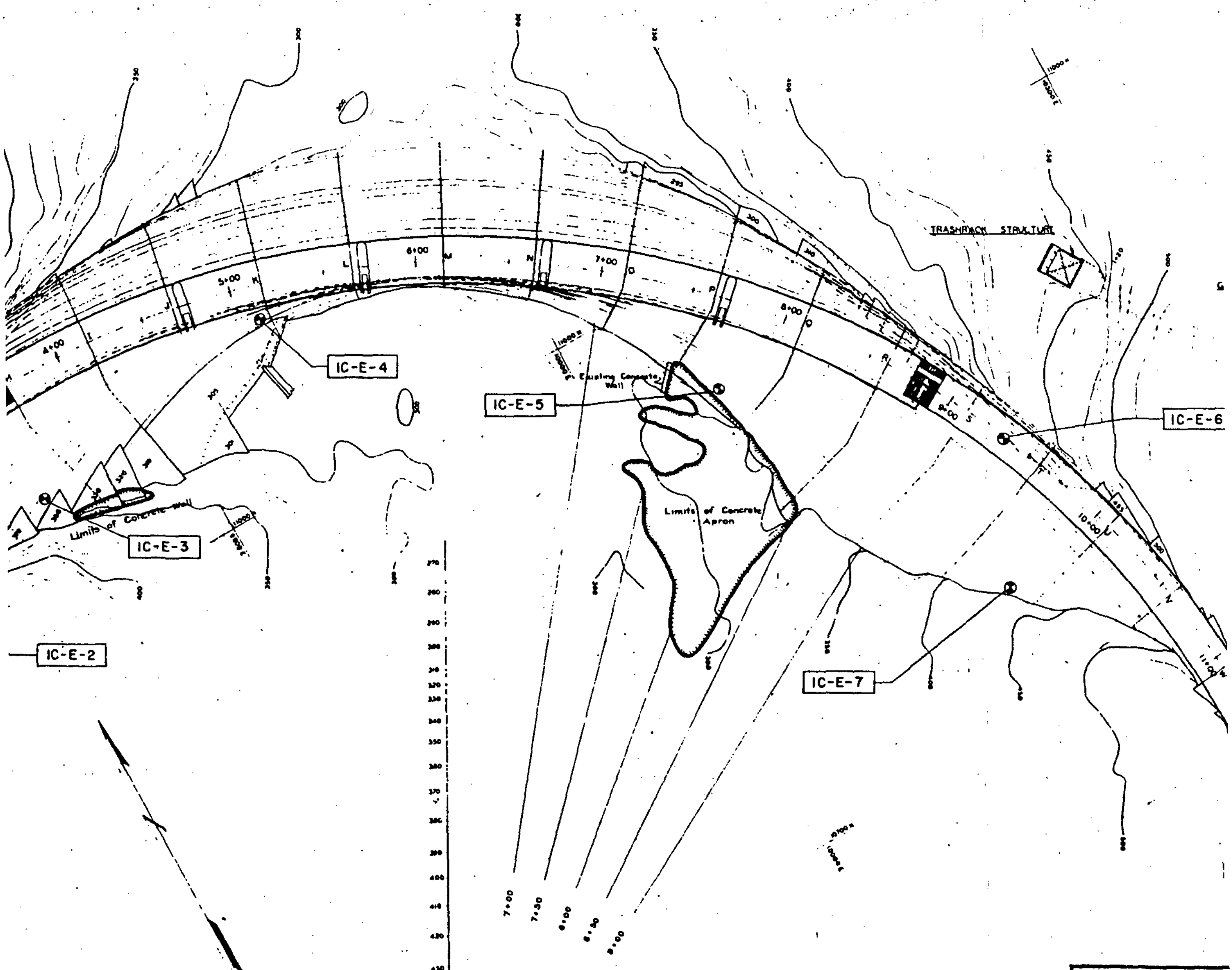
ENGLEBRIGHT DAM
ELEVATION

DESIGNED: J. HESS	
DRAWN: B. SIMPSON	
CHECKED: R. TREAT	

SUBMITTED	DATE APPROVED	SCALE AS SHOWN	SPEC. No.
Justin B. Moses	19 Feb 82	SHEET 2	FILE No. 8-1-844



IC-E-1
IC-E-2
IC-E-3
IC-E-4
Limits of Concrete Wall
CONCRETE CORE DRILL SITES
IC-A
TO
IC-DD



NOTES:

1. No power is available at Englebright Dam.
2. Drill fluid may not be disposed of into reservoir or river.

ITES IC-A TO IC-DD

GRAPHIC SCALE

FIGURE D-3
CORES, SUPP
GRAM - PLAN

REVISION DATE

DESIGNED J. HESS
DRAWN
CHECKED J.PETERSEN
SUBMITTED

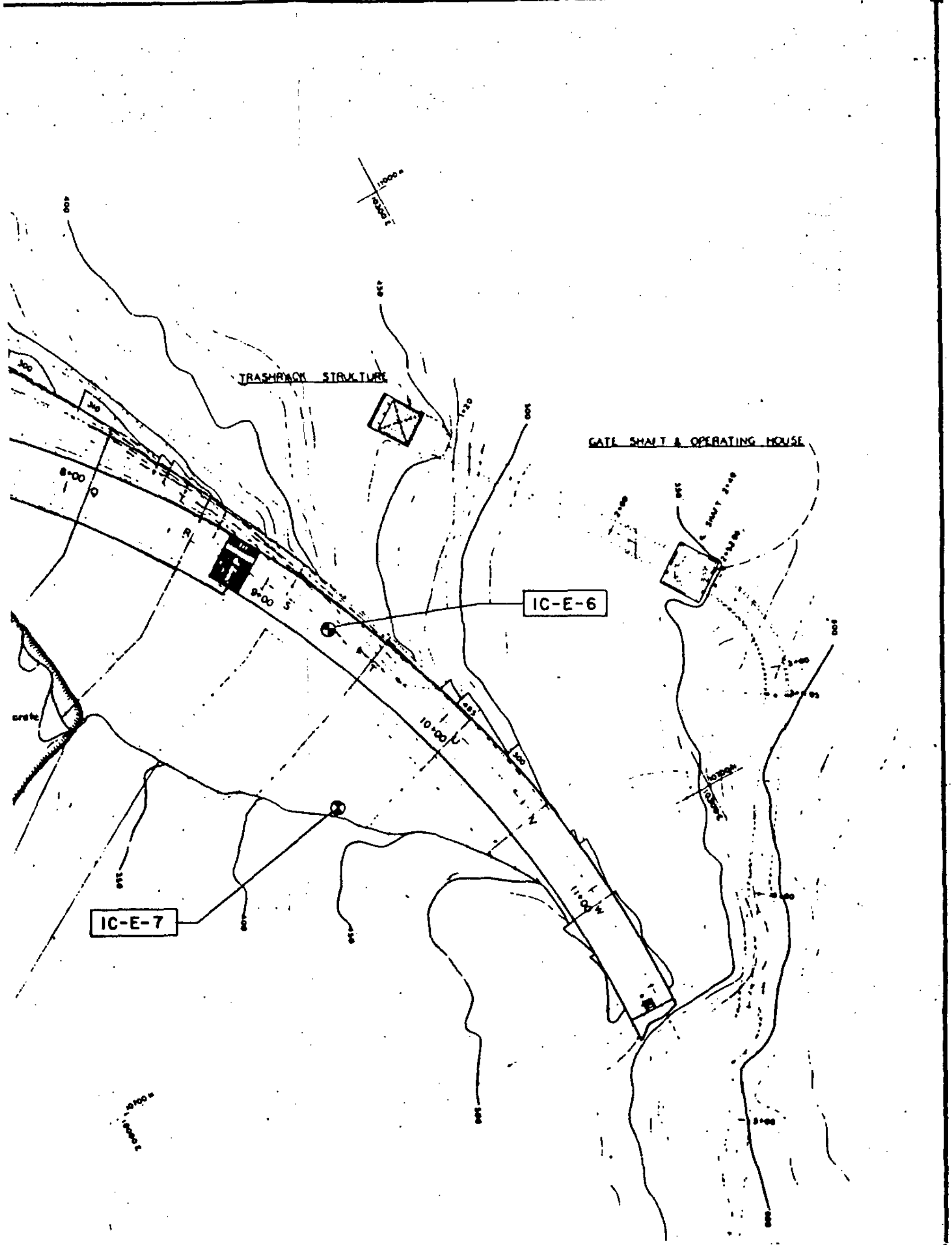
ENGLE
ROC

SIGNATURES AFFIXED BELO
APPROVAL OF ALL DRAWIN

APPROVAL RECOMMENDED

PREPARED UNDER THE DIRECTION OF

2



lable at Englebright Dam.
be disposed of into

SCALE

FIGURE D-3 LOCATIONS OF CONCRETE CORES, SUPPLEMENTAL TESTING PROGRAM - PLAN VIEW

REVISION | DATE | DESCRIPTION | BY

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

DESIGNED J. HESS
DRAWN
CHECKED J. PETERSEN
SUBMITTED
CHIEF, S MAT. SECT.

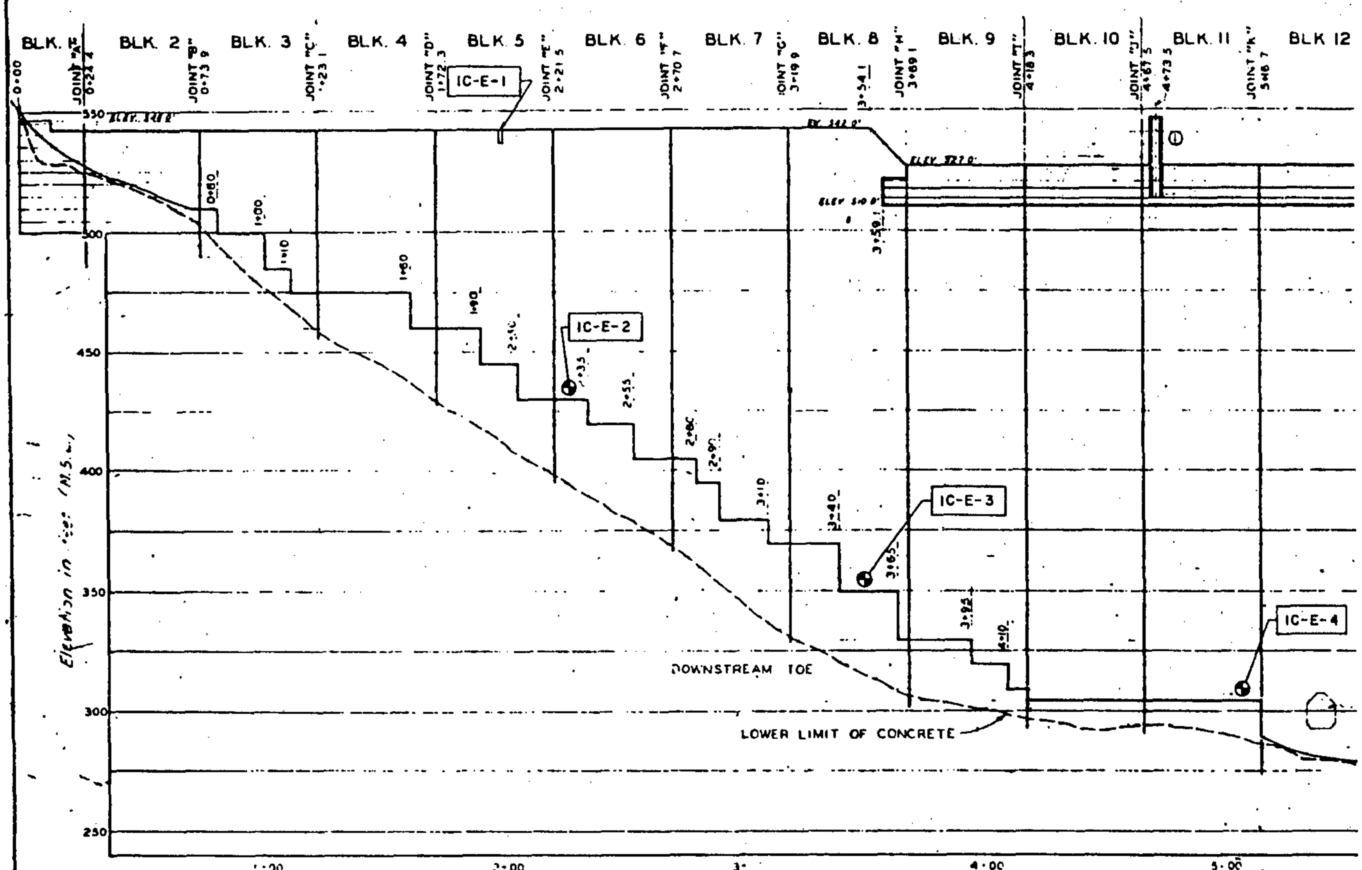
ENGLEBRIGHT, PINE FLAT, AND FOLSOM DAMS, CALIFORNIA
CONCRETE CORE DRILLING,
ROCK CORE DRILLING AND TESTING.
ENGLEBRIGHT DAM
PLAN VIEW

SIGNATURES AFFIXED BELOW INDICATE OFFICIAL RECOMMENDATION AND APPROVAL OF ALL DRAWINGS IN THIS SET AS INDEXED ON THIS SHEET

APPROVAL RECOMMENDED | APPROVED | DATE

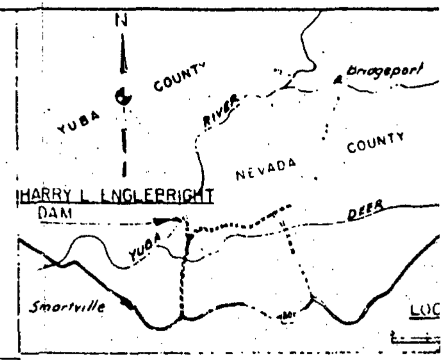
PREPARED UNDER THE DIRECTION OF

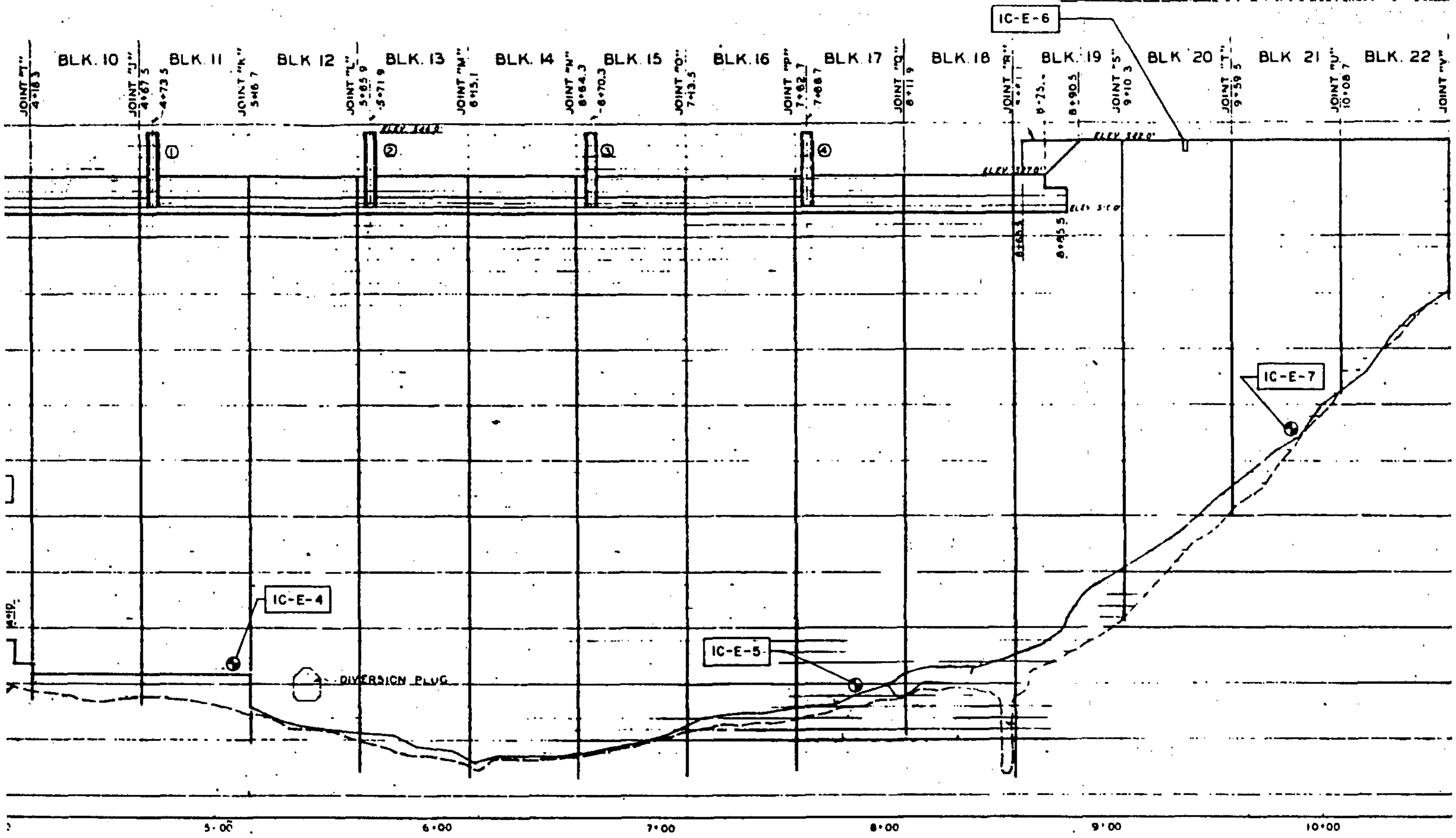
SCALE: 1" = 30'-0" | SPEC. No.
SHEET 1 | FILE No.



BLK. 1
BLK. 2
BLK. 3
BLK. 4
BLK. 5
BLK. 6
BLK. 7
BLK. 8
BLK. 9
BLK. 10
BLK. 11
BLK 12
IC-E-1
IC-E-2
IC-E-3
IC-E-4
DOWNSTREAM TOE
LOWER LIMIT OF CONCRETE
550
500
450
400
350
300
250
4+00
5+00

DEVELOPE





DEVELOPED PROFILE LOOKING UPSTREAM

SCALE 1"=30'-0"

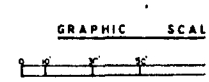


FIGURE D-4 LOCATI
CORES, SUPPLEMENT
GRAM - ELEVATION

NOTE:

1. CONCRETE CORE DRILL SITES IC-A TO IC-DD

J. HESS	ENGLEBRIGHT, PINE
	CONCRE
	ROCK CORE
J. PETERSEN	ENGL

CORPS OF ENGINEERS U. S. ARMY

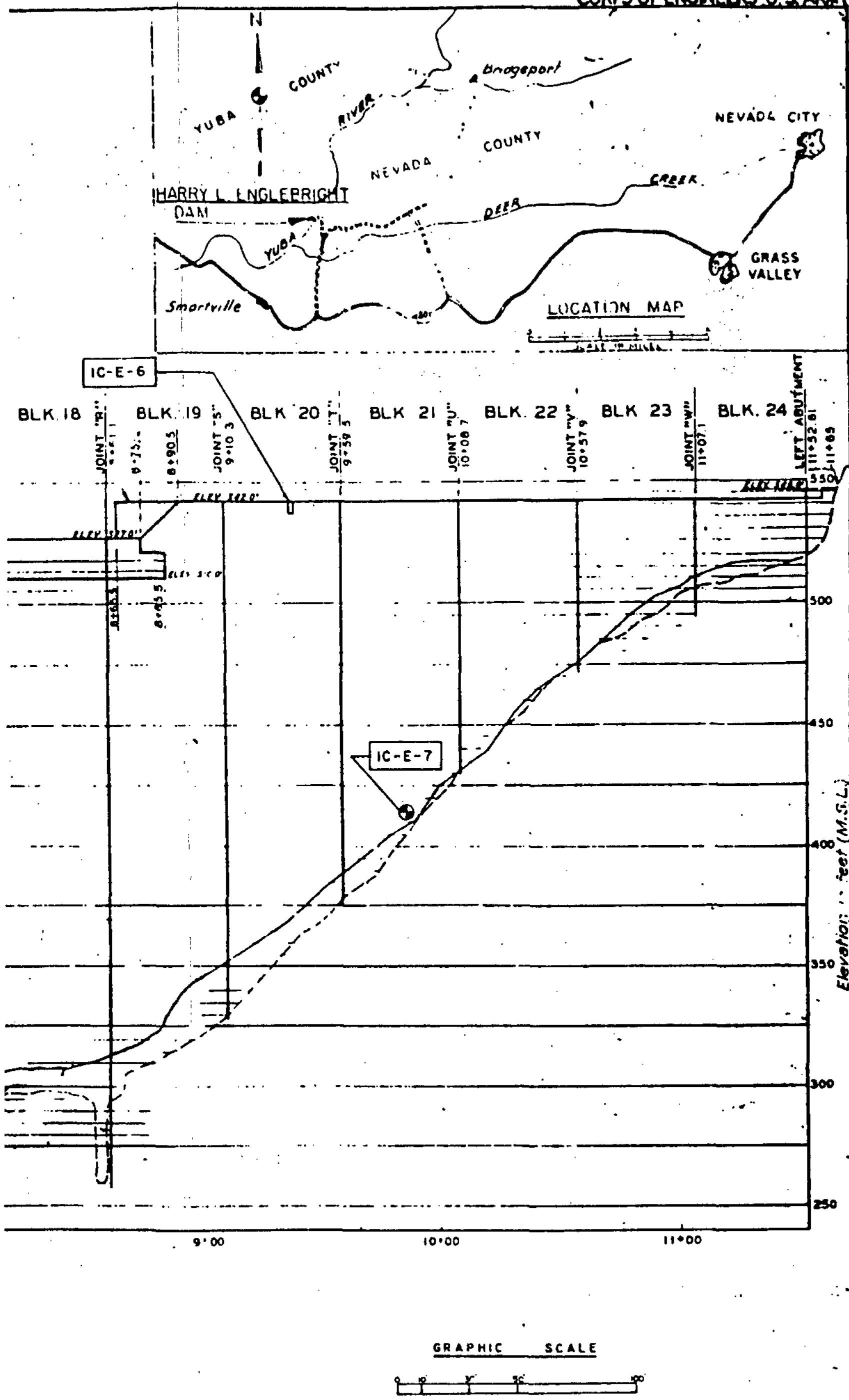


FIGURE D-4 LOCATIONS OF CONCRETE CORES, SUPPLEMENTAL TESTING PROGRAM - ELEVATION VIEW

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

ENGLEBRIGHT, PINE FLAT, AND FOLSOM DAMS, CALIFORNIA

CONCRETE CORE DRILLING,
ROCK CORE DRILLING AND TESTING

ENGLEBRIGHT DAM
ELEVATION

J. HESS
J. PETERSEN

SCALE AS SHOWN
SHEET 2

TO IC-DD

3

APPENDIX E

ADDITIONAL EARTHQUAKE RESPONSE RESULTS

TABLE E-1:SUMMARY OF THE FIRST FIVE PEAK TENSILE STRESSES* (IN DESCENDIN
RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RES
HYDROSTATIC PRESSURE ARE INCLUDED.

QUAKE 1		CASE 1 $\eta = 0.10$ $\alpha = 1.00$					CASE 2 $\eta = 0.10$ $\alpha = 0.90$					CASE 3 $\eta = 0.10$ $\alpha = 0.75$					CASI $\eta =$ ($\alpha =$ (		
ARCH STRESS	DAM ELEMENT NUMBER	9	9	8	8	9	9	9	8	8	9	9	9	8	9	9	9	9	8
	STRESS LOCATION NUMBER†	4	1	1	4	3	4	1	1	4	3	4	1	1	3	2	4	1	1
	PEAK TENSILE STRESS *(PSI)*	1295	1270	1206	1060	1000	1091	1076	1051	958	926	1052	1026	983	884	856	932	897	8
	TIME OF OCCURRENCE *(SEC.)*	7.70	7.70	5.04	5.04	7.70	7.24	7.24	5.04	5.04	7.24	7.24	7.24	7.26	7.24	7.24	7.24	7.24	7.2
CANTILEVER STRESS	DAM ELEMENT NUMBER	56	56	55	56	52	56	56	56	55	52	56	56	56	52	55	56	56	5
	STRESS LOCATION NUMBER†	2	3	2	1	2	2	3	1	2	2	2	3	1	2	2	3	2	
	PEAK TENSILE STRESS *(PSI)*	785	775	753	724	707	712	699	655	640	638	639	628	586	570	567	585	577	5
	TIME OF OCCURRENCE *(SEC.)*	7.58	7.58	4.92	7.58	7.58	7.58	7.58	7.58	4.92	7.58	7.58	7.58	7.58	7.58	7.56	7.56	7.56	7.
PRINCIPAL STRESS	DAM ELEMENT NUMBER	9	9	8	8	9	9	9	8	8	9	9	9	8	9	9	9	9	
	STRESS LOCATION NUMBER†	4	1	1	4	3	4	1	1	4	3	4	1	1	3	2	4	1	
	PEAK TENSILE STRESS *(PSI)*	1297	1270	1206	1063	1001	1091	1076	1051	959	927	1052	1026	983	884	857	933	897	8
	TIME OF OCCURRENCE *(SEC.)*	7.70	7.70	5.04	5.04	7.70	7.24	7.24	5.04	5.04	7.24	7.24	7.24	7.26	7.24	7.24	7.24	7.24	7.

*PEAK TENSILE STRESS CORRESPONDS TO THE PEAK VALUE IN THE STRESS TIME-HISTORY RESPONSE.

†EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE

ILE STRESSES* (IN DESCENDING ORDER) COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEM
INDATION ROCK WITH FULL RESERVOIR, IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT

CASE 3 η = 0.10 α = 0.75				CASE 4 η = 0.10 α = 0.50					CASE 5 η = 0.14 α = 1.00					CASE 6 η = 0.14 α = 0.90					CASE 7 η = 0.14 α = 0.75				
9	8	9	9	9	9	8	9	8	9	9	8	9	8	9	9	8	8	9	9	9	8	9	8
1	1	3	2	4	1	1	3	4	4	1	1	3	4	4	1	1	4	3	4	1	1	3	4
026	983	884	856	932	897	887	776	768	1165	1079	990	910	853	917	906	880	801	783	919	872	867	756	744
.24	7.26	7.24	7.24	7.24	7.24	7.26	7.24	7.26	6.78	6.78	6.76	6.78	6.76	7.24	7.24	5.04	5.04	7.24	7.26	7.24	7.26	7.26	7.26
56	56	52	55	56	56	55	56	52	56	55	56	50	51	56	55	56	50	51	56	56	55	50	56
3	1	2	2	3	2	2	1	2	3	2	2	2	3	3	2	2	2	3	2	3	2	2	1
628	586	570	567	585	577	544	533	526	687	679	660	651	637	593	582	573	558	532	520	511	438	483	482
.58	7.58	7.58	7.56	7.56	7.56	7.56	7.56	7.56	4.92	4.92	4.92	4.92	4.92	4.92	4.92	7.58	4.92	4.92	7.58	7.56	4.92	4.92	5.38
9	8	9	9	9	9	8	9	8	9	9	8	9	8	9	9	8	8	9	9	9	8	9	8
1	1	3	2	4	1	1	3	4	4	1	1	3	4	4	1	1	4	3	4	1	1	3	4
1026	983	884	857	933	897	887	776	769	1167	1079	992	912	855	917	907	880	803	783	919	872	868	756	746
.24	7.26	7.24	7.24	7.24	7.24	7.26	7.24	7.26	6.78	6.78	6.76	6.78	6.76	7.24	7.24	5.04	5.04	7.24	7.26	7.24	7.26	7.26	7.26

HISTORY RESPONSE.
DINTS) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

2

RE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS. STRESS
N. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND

CASE 6 η = 0.14 α = 0.90				CASE 7 η = 0.14 α = 0.75					CASE 8 η = 0.14 α = 0.50				
9	8	8	9	9	9	8	9	8	9	8	9	8	9
1	1	4	3	4	1	1	3	4	4	1	1	4	3
906	880	801	783	919	872	867	756	744	812	786	772	682	670
7.24	5.04	5.04	7.24	7.26	7.24	7.26	7.26	7.26	7.26	7.26	7.24	7.26	7.24
55	56	50	51	56	56	55	50	56	56	56	55	56	52
2	2	2	3	2	3	2	2	1	3	2	2	1	2
582	573	558	532	520	511	438	483	482	494	486	460	446	438
4.92	7.58	4.92	4.92	7.58	7.56	4.92	4.92	5.38	7.56	7.56	7.56	7.56	7.56
9	8	8	9	9	9	8	9	8	9	8	9	8	9
1	1	4	3	4	1	1	3	4	4	1	1	4	3
907	880	803	783	919	872	868	756	746	812	786	772	683	670
7.24	5.04	5.04	7.24	7.26	7.24	7.26	7.26	7.26	7.26	7.26	7.24	7.26	7.24

3

TABLE E-2: SUMMARY OF THE FIRST FIVE PEAK TENSILE STRESSES* (IN DESCENDING FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR, IS DUE TO QU ARE INCLUDED.

QUAKE I		CASE 1 $\eta = 0.10$ $\alpha = 1.00$					CASE 2 $\eta = 0.10$ $\alpha = 0.90$					CASE 3 $\eta = 0.10$ $\alpha = 0.75$					CASE $\eta = 0.$ $\alpha = 0$		
ARCH STRESS	DAM NODAL POINT NUMBER	304	303	305	302	319	303	304	302	305	319	303	304	305	319	302	303	304	305
	PEAK TENSILE STRESS (PSI)	1629	1567	1432	1355	1304	1319	1235	1209	1185	1095	1230	1194	1117	1064	1049	1106	1054	968
	TIME OF OCCURRENCE (SEC.)	7.70	5.04	7.92	5.04	5.04	5.04	7.24	5.04	7.24	5.04	7.26	7.24	7.24	7.26	7.26	7.26	7.24	5.48
CANTILEVER STRESS	DAM NODAL POINT NUMBER	484	486	472	487	481	484	486	481	472	487	484	486	487	481	476	486	484	487
	PEAK TENSILE STRESS (PSI)	1497	1486	1476	1445	1444	1382	1334	1330	1329	1327	1257	1215	1210	1208	1175	1133	1117	1083
	TIME OF OCCURRENCE (SEC.)	7.58	4.92	4.92	7.58	7.58	7.58	7.58	7.58	4.92	7.58	7.58	7.58	7.58	7.58	7.58	7.56	7.58	7.56
PRINCIPAL STRESS	DAM NODAL POINT NUMBER	304	472	303	484	481	484	472	481	476	487	484	481	472	476	303	484	486	481
	PEAK TENSILE STRESS (PSI)	1630	1572	1567	1555	1507	1440	1410	1393	1360	1346	1314	1269	1249	1237	1230	1172	1136	1131
	TIME OF OCCURRENCE (SEC.)	7.70	4.92	5.04	7.58	7.58	7.58	4.92	7.58	7.58	7.58	7.58	7.58	5.38	7.58	7.26	7.58	7.56	7.58

*PEAK TENSILE STRESS CORRESPONDS TO THE PEAK VALUE IN THE STRESS TIME-HISTORY RESPONSE.

TRESSES* (IN DESCENDING ORDER) COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS. STRESS RESPO
RESERVOIR, IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTA

	ASE 3 = 0.10 = 0.75			CASE 4 $\eta = 0.10$ $\alpha = 0.50$					CASE 5 $\eta = 0.14$ $\alpha = 1.00$					CASE 6 $\eta = 0.14$ $\alpha = 0.90$					CASE 7 $\eta = 0.14$ $\alpha = 0.75$					
	305	319	302	303	304	305	302	319	304	303	319	305	302	303	304	302	305	319	303	304	305	319	302	303
	1117	1064	1049	1106	1054	968	965	949	1417	1369	1155	1147	1086	1104	1035	1013	998	924	1086	1032	949	938	924	981
I	7.24	7.26	7.26	7.26	7.24	5.48	7.26	7.26	6.78	6.78	6.78	6.78	5.04	5.04	7.24	5.04	7.24	7.26	7.26	7.26	7.24	7.26	7.26	7.2(
5	487	481	476	486	484	487	481	472	472	486	484	487	481	472	486	484	487	481	472	484	486	481	476	472
5	1210	1208	1175	1133	1117	1089	1072	1042	1391	1344	1258	1247	1200	1222	1194	1141	1107	1102	1065	1051	1040	1025	1020	99
58	7.58	7.58	7.58	7.56	7.58	7.56	7.58	5.38	4.92	4.92	4.92	4.92	4.92	4.92	4.92	7.58	4.92	5.38	4.92	7.58	5.38	5.38	5.38	6.(
81	472	476	303	484	486	481	472	303	472	304	303	486	484	472	486	484	481	476	472	484	303	481	476	4
269	1249	1237	1230	1172	1136	1131	1109	1106	1477	1417	1376	1349	1280	1295	1197	1191	1149	1128	1128	1100	1087	1060	1047	1
.58	5.38	7.58	7.26	7.58	7.56	7.58	5.38	7.26	4.92	6.78	6.78	4.92	4.92	4.92	4.92	7.58	7.58	5.38	4.92	7.58	7.26	7.58	5.38	6

STORY RESPONSE.

2

DAL POINTS OF THE 3-D SOLID DAM ELEMENTS. STRESS RESPONSE, COMPUTED
TIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE

14 00		CASE 6 $\eta = 0.14$ $\alpha = 0.90$					CASE 7 $\eta = 0.14$ $\alpha = 0.75$					CASE 8 $\eta = 0.14$ $\alpha = 0.50$				
305	302	303	304	302	305	319	303	304	305	319	302	303	304	302	319	305
1147	1086	1104	1035	1013	998	924	1086	1032	949	938	924	981	906	852	840	826
5.78	5.04	5.04	7.24	5.04	7.24	7.26	7.26	7.26	7.24	7.26	7.26	7.26	7.26	7.26	7.26	7.24
487	481	472	486	484	487	481	472	484	486	481	476	472	486	484	487	481
1247	1200	1222	1194	1141	1107	1102	1065	1051	1040	1025	1020	991	987	965	942	926
.92	4.92	4.92	4.92	7.58	4.92	5.38	4.92	7.58	5.38	5.38	5.38	6.64	7.56	6.64	7.56	6.64
486	484	472	486	484	481	476	472	484	303	481	476	472	484	486	303	481
349	1280	1295	1197	1191	1149	1128	1128	1100	1087	1060	1047	1044	994	989	981	956
.92	4.92	4.92	4.92	7.58	7.58	5.38	4.92	7.58	7.26	7.58	5.38	6.64	7.58	7.56	7.26	7.58

3

TABLE E-3: SUMMARY OF THE FIRST FIVE PEAK TENSILE STRESSES* (IN DESCENDIN
COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR, IS
PRESSURE ARE INCLUDED.

QUAKE 2		CASE 9 $\eta = 0.10$ $\alpha = 1.00$					CASE 10 $\eta = 0.10$ $\alpha = 0.90$					CASE 11 $\eta = 0.10$ $\alpha = 0.75$					CASE η = (α = (		
ARCH STRESS	DAM ELEMENT NUMBER	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8
	STRESS LOCATION NUMBER†	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1
	PEAK TENSILE STRESS *(PSI)*	1758	1708	1528	1469	1423	1831	1744	1635	1519	1447	1664	1588	1482	1378	1315	1401	1341	12
	TIME OF OCCURRENCE *(SEC.)*	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.
CANTILEVER STRESS	DAM ELEMENT NUMBER	56	56	56	52	57	56	56	56	52	57	56	56	56	52	57	56	56	5
	STRESS LOCATION NUMBER†	2	3	1	2	4	2	3	1	2	4	2	3	1	2	4	2	3	1
	PEAK TENSILE STRESS *(PSI)*	1053	1001	989	974	965	1035	988	973	961	940	926	883	867	855	837	762	725	71
	TIME OF OCCURRENCE *(SEC.)*	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.
PRINCIPAL STRESS	DAM ELEMENT NUMBER	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8
	STRESS LOCATION NUMBER†	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1
	PEAK TENSILE STRESS *(PSI)*	1758	1708	1530	1470	1424	1831	1744	1636	1519	1447	1665	1588	1483	1378	1315	1402	1341	12
	TIME OF OCCURRENCE *(SEC.)*	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.

*PEAK TENSILE STRESS CORRESPONDS TO THE PEAK VALUE IN THE STRESS TIME-HISTORY RESPONSE.

†EACH 3-D SOLID DAM ELEMENT HAS 8 STRESS LOCATIONS (GAUSS QUADRATURE POINTS) INSIDE THE ELEMENT. THE LOCA

LE STRESSES* (IN DESCENDING ORDER) COMPUTED AT GAUSS QUADRATURE POINTS INSIDE THE 3-D SOLID DAM ELEME
CK WITH FULL RESERVOIR, IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM

CASE 11 η = 0.10 α = 0.75				CASE 12 η = 0.10 α = 0.50					CASE 13 η = 0.14 α = 1.00					CASE 14 η = 0.14 α = 0.90					CASE 15 η = 0.14 α = 0.75					
9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	
1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	
588	1482	1378	1315	1401	1341	1240	1155	1104	1434	1394	1247	1197	1160	1546	1472	1387	1283	1222	1415	1351	1264	1172	1119	1.
.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4
56	56	52	57	56	56	56	52	51	56	51	56	56	52	56	56	56	52	57	56	56	56	52	57	
3	1	2	4	2	3	1	2	3	2	3	3	1	2	2	3	1	2	4	2	3	1	2	4	
383	867	855	837	762	725	710	697	691	825	817	810	769	753	851	813	797	785	768	767	733	716	702	689	6
.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	3.12	4.78	3.12	3.14	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4
9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	9	9	8	9	9	
1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	4	1	1	3	2	
588	1483	1378	1315	1402	1341	1241	1155	1104	1435	1394	1248	1198	1160	1546	1472	1388	1283	1222	1415	1351	1265	1172	1119	1
.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4

RY RESPONSE.
) INSIDE THE ELEMENT. THE LOCATIONS ARE NUMBERED 1 TO 8.

2

RE POINTS INSIDE THE 3-D SOLID DAM ELEMENTS. STRESS RESPONSE,
C STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC

CASE 14 $\eta = 0.14$ $\alpha = 0.90$				CASE 15 $\eta = 0.14$ $\alpha = 0.75$					CASE 16 $\eta = 0.14$ $\alpha = 0.50$				
9	8	9	9	9	9	8	9	9	9	9	8	9	9
1	1	3	2	4	1	1	3	2	4	1	1	3	2
1472	1387	1283	1222	1415	1351	1264	1172	1119	1202	1153	1060	990	949
4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66
56	56	52	57	56	56	56	52	57	56	50	51	55	56
3	1	2	4	2	3	1	2	4	2	2	3	2	3
813	797	785	768	767	733	716	702	689	637	628	625	612	605
4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	3.12	3.12	3.12	4.78
9	8	9	9	9	9	8	9	9	9	9	8	9	9
1	1	3	2	4	1	1	3	2	4	1	1	3	2
1472	1388	1283	1222	1415	1351	1265	1172	1119	1202	1153	1062	990	949
4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66

3

TABLE E-4: SUMMARY OF THE FIRST FIVE PEAK TENSILE STRESSES* (IN DESCENDING
THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR, IS DUE TO QUAKE 2
INCLUDED.

	QUAKE 2	CASE 9 η = 0.10 α = 1.00					CASE 10 η = 0.10 α = 0.90					CASE 11 η = 0.10 α = 0.75					CASE 1 η = 0. α = 0.		
ARCH STRESS	DAM NODAL POINT NUMBER	304	303	305	319	320	303	304	319	305	302	303	304	319	305	302	304	303	305
	PEAK TENSILE STRESS (*PSI*)	2032	1983	1830	1749	1603	2124	2110	1856	1832	1701	1929	1920	1685	1671	1540	1624	1623	1417
	TIME OF OCCURRENCE (*SEC.*)	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66
CANTILEVER STRESS	DAM NODAL POINT NUMBER	484	481	476	487	486	484	481	476	487	486	484	481	476	487	486	472	484	481
	PEAK TENSILE STRESS (*PSI*)	2015	1960	1935	1861	1839	1973	1921	1898	1830	1810	1784	1734	1710	1656	1640	1499	1497	1451
	TIME OF OCCURRENCE (*SEC.*)	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	3.12	4.78	4.7
PRINCIPAL STRESS	DAM NODAL POINT NUMBER	484	304	481	476	303	303	304	484	481	476	303	304	484	481	476	303	304	472
	PEAK TENSILE STRESS (*PSI*)	2062	2033	2012	1990	1987	2126	2110	2022	1974	1954	1931	1921	1829	1783	1762	1625	1624	1574
	TIME OF OCCURRENCE (*SEC.*)	4.78	4.66	4.78	4.78	4.66	4.66	4.66	4.78	4.78	4.78	4.66	4.66	4.78	4.78	4.78	4.66	4.66	3.1

*PEAK TENSILE STRESS CORRESPONDS TO THE PEAK VALUE IN THE STRESS TIME-HISTORY RESPONSE.

ILE STRESSES* (IN DESCENDING ORDER) COMPUTED AT NODAL POINTS OF THE 3-D SOLID DAM ELEMENTS. STRESS
L RESERVOIR, IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROST

	CASE 11 η = 0.10 α = 0.75				CASE 12 η = 0.10 α = 0.50					CASE 13 η = 0.14 α = 1.00					CASE 14 η = 0.14 α = 0.90					CASE 15 η = 0.14 α = 0.75				
	304	319	305	302	304	303	305	319	302	304	303	305	319	320	303	304	319	305	302	303	304	319	305	302
	1920	1685	1671	1540	1624	1623	1417	1413	1285	1658	1616	1499	1424	1310	1790	1774	1564	1547	1444	1636	1628	1429	1425	131
	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.6
	481	476	487	486	472	484	481	476	487	472	484	481	486	476	484	472	481	476	487	484	472	481	476	487
4	1734	1710	1656	1640	1499	1497	1451	1428	1390	1735	1618	1569	1551	1544	1650	1603	1603	1581	1534	1506	1492	1460	1437	140
8	4.78	4.78	4.78	4.78	3.12	4.78	4.78	4.78	4.78	3.12	4.78	4.78	3.12	4.78	4.78	3.12	4.78	4.78	4.78	4.78	3.12	4.78	4.78	4.7
3	304	484	481	476	303	304	472	484	481	472	304	484	303	481	303	304	484	472	481	303	304	472	484	48
1	1921	1829	1783	1762	1625	1624	1574	1537	1495	1820	1659	1658	1619	1612	1791	1774	1692	1682	1650	1638	1629	1564	1545	150
6	4.66	4.78	4.78	4.78	4.66	4.66	3.12	4.78	4.78	3.12	4.66	4.78	4.66	4.78	4.66	4.66	4.78	3.12	4.78	4.66	4.66	3.12	4.78	4.7

-HISTORY RESPONSE.

S OF THE 3-D SOLID DAM ELEMENTS. STRESS RESPONSE, COMPUTED FOR
)UE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE

CASE 14 $\eta = 0.14$ $\alpha = 0.90$					CASE 15 $\eta = 0.14$ $\alpha = 0.75$					CASE 16 $\eta = 0.14$ $\alpha = 0.50$				
303	304	319	305	302	303	304	319	305	302	304	303	305	319	302
790	1774	1564	1547	1444	1636	1628	1429	1425	1312	1392	1386	1224	1206	1097
.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66	4.66
484	472	481	476	487	484	472	481	476	487	472	484	481	476	486
650	1603	1603	1581	1534	1506	1492	1460	1437	1401	1375	1278	1237	1214	1204
.78	3.12	4.78	4.78	4.78	4.78	3.12	4.78	4.78	4.78	3.12	4.78	4.78	4.78	3.12
303	304	484	472	481	303	304	472	484	481	472	304	303	484	481
791	1774	1692	1682	1650	1638	1629	1564	1545	1503	1439	1392	1388	1314	1275
.66	4.66	4.78	3.12	4.78	4.66	4.66	3.12	4.78	4.78	3.12	4.66	4.66	4.78	4.78

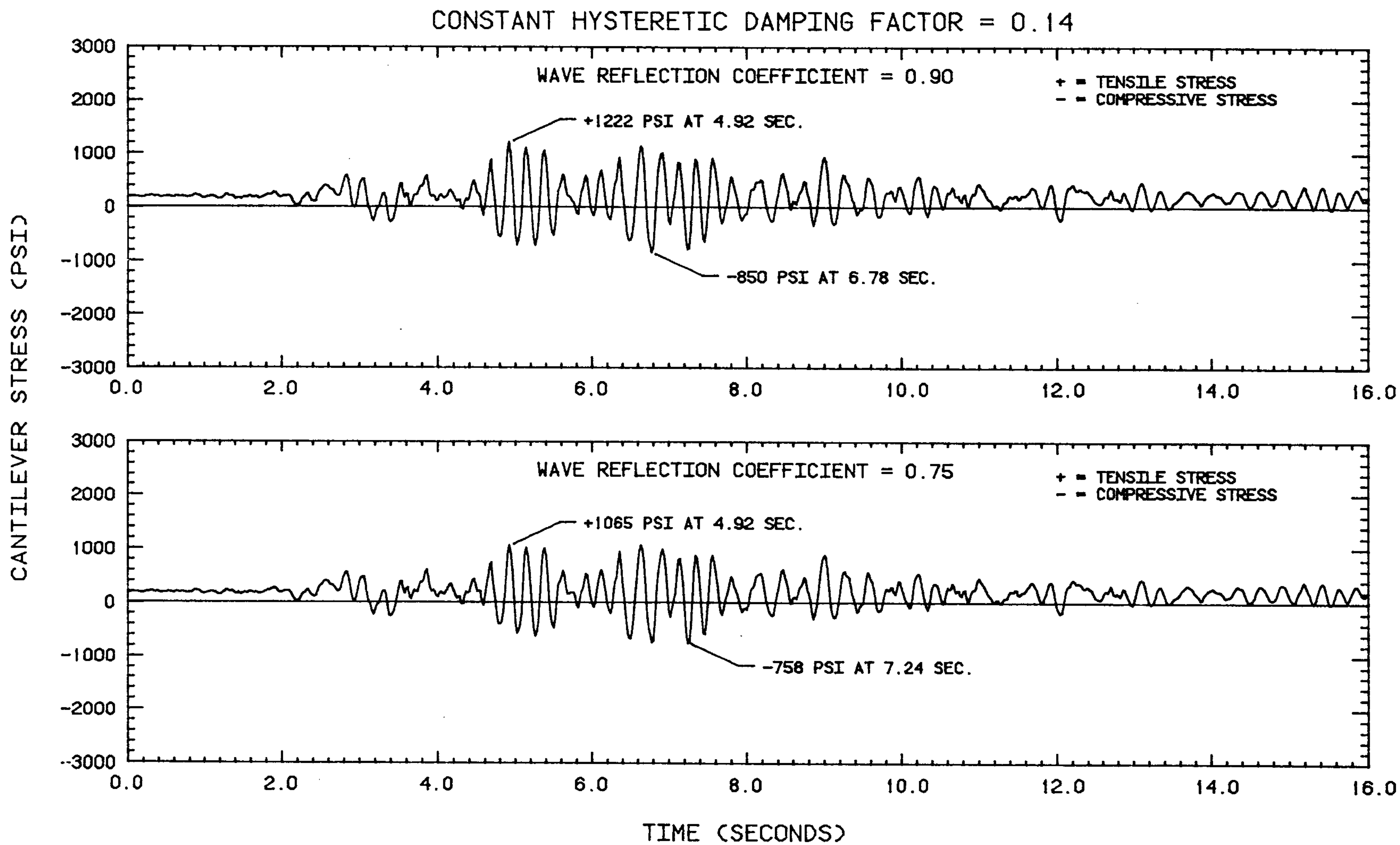


FIGURE E-1 CANTILEVER STRESS RESPONSE AT NODAL POINT 472 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

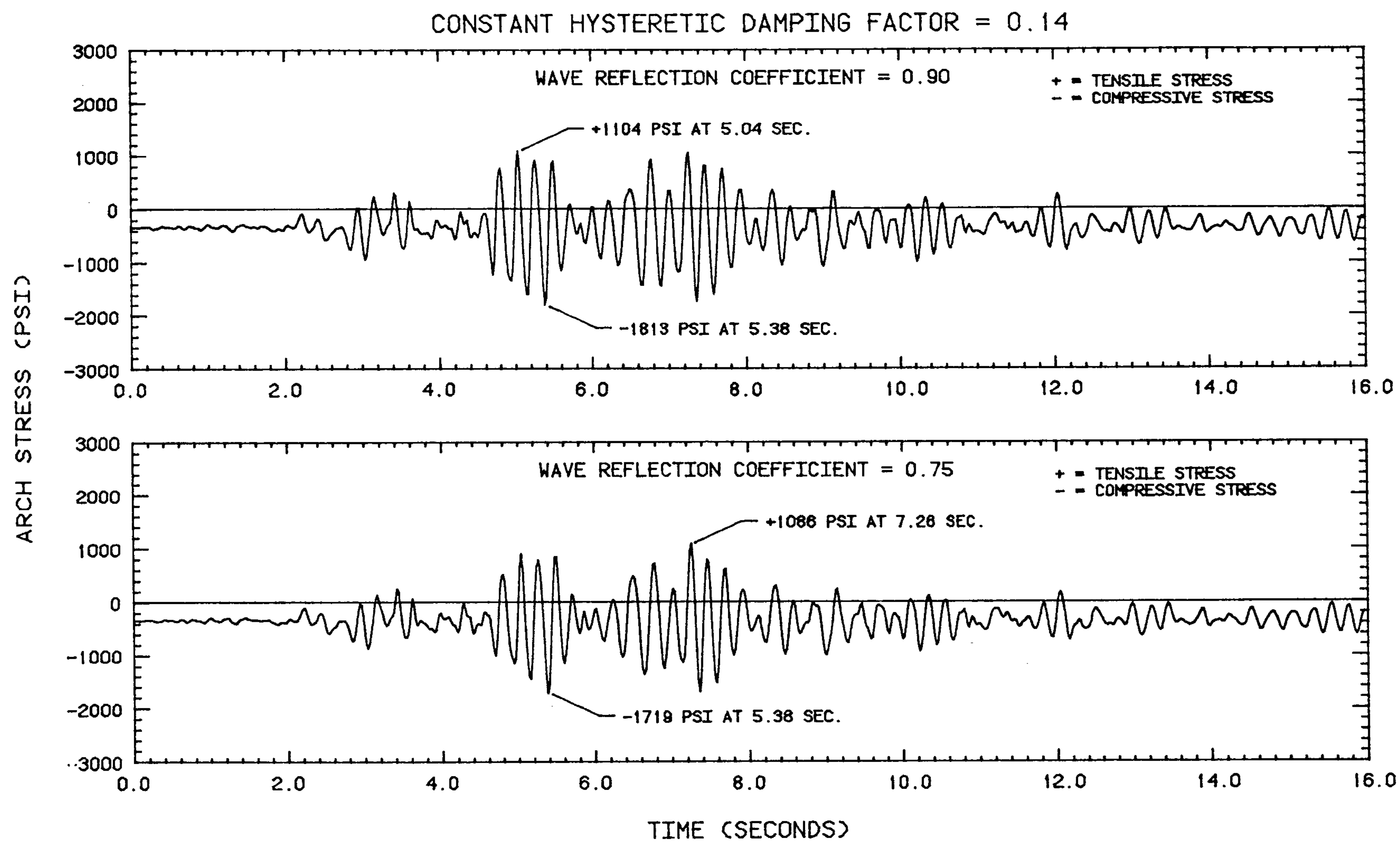


FIGURE E-2 ARCH STRESS RESPONSE AT NODAL POINT 303 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

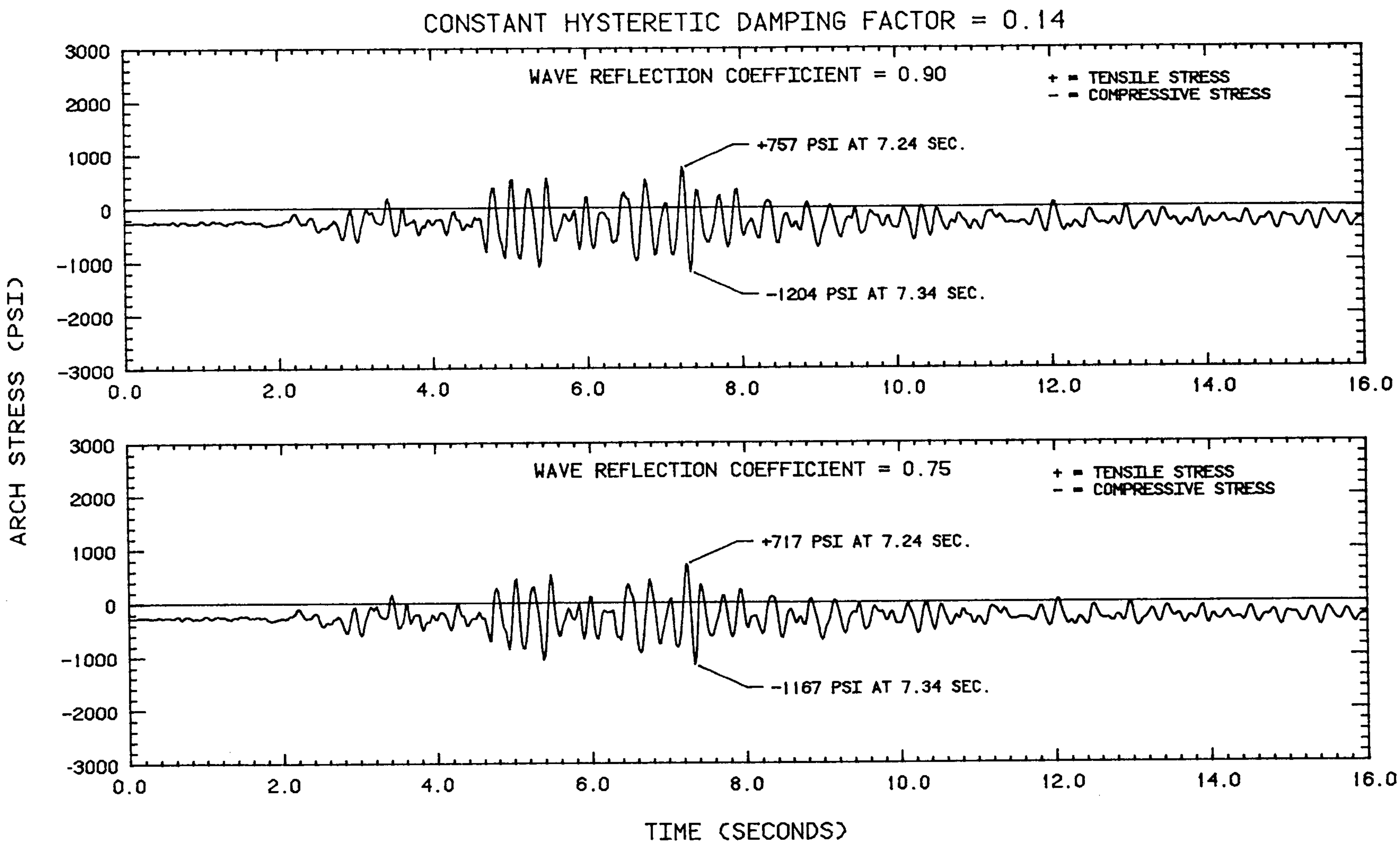


FIGURE E-3 ARCH STRESS RESPONSE AT NODAL POINT 22 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

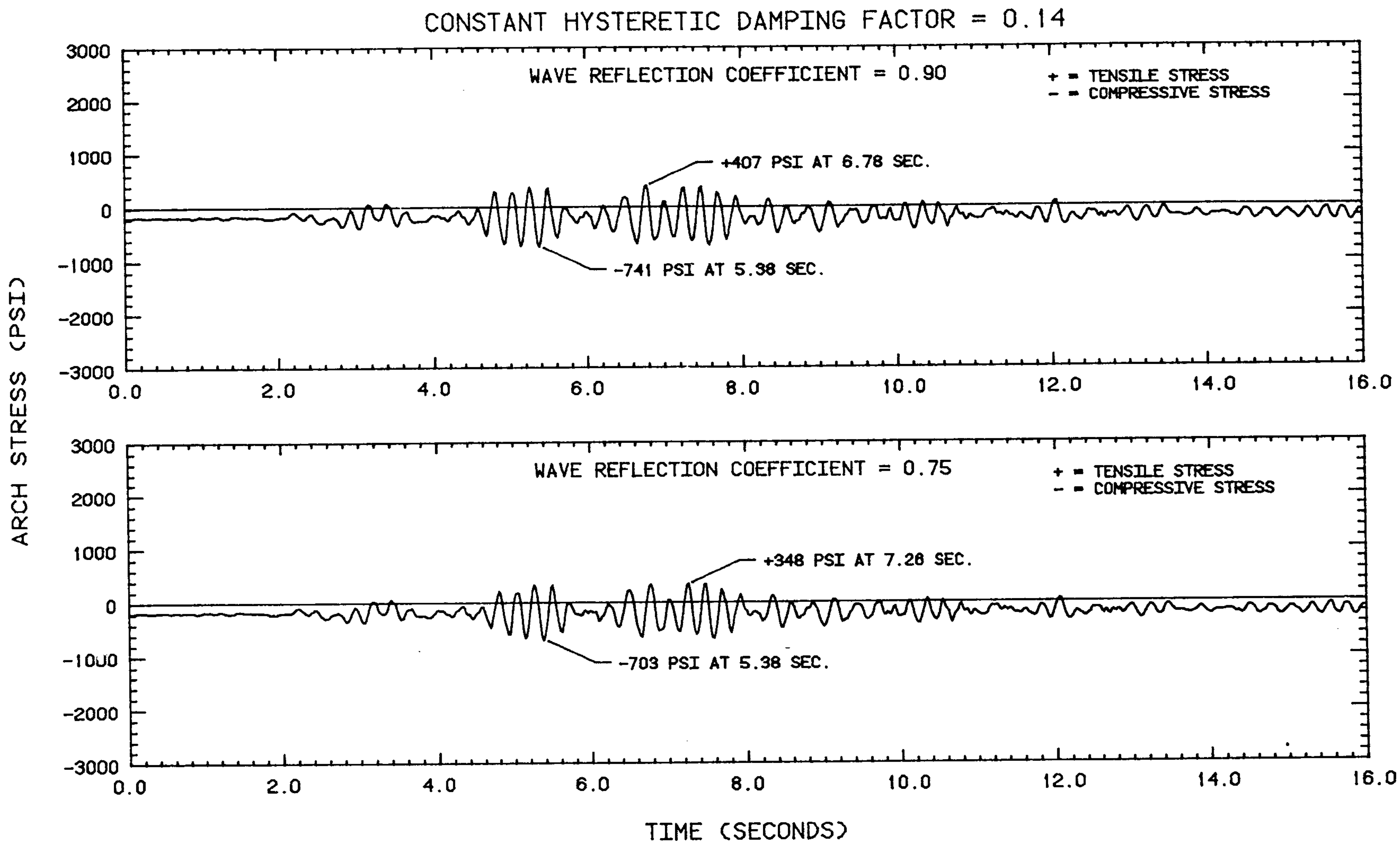


FIGURE E-4 ARCH STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

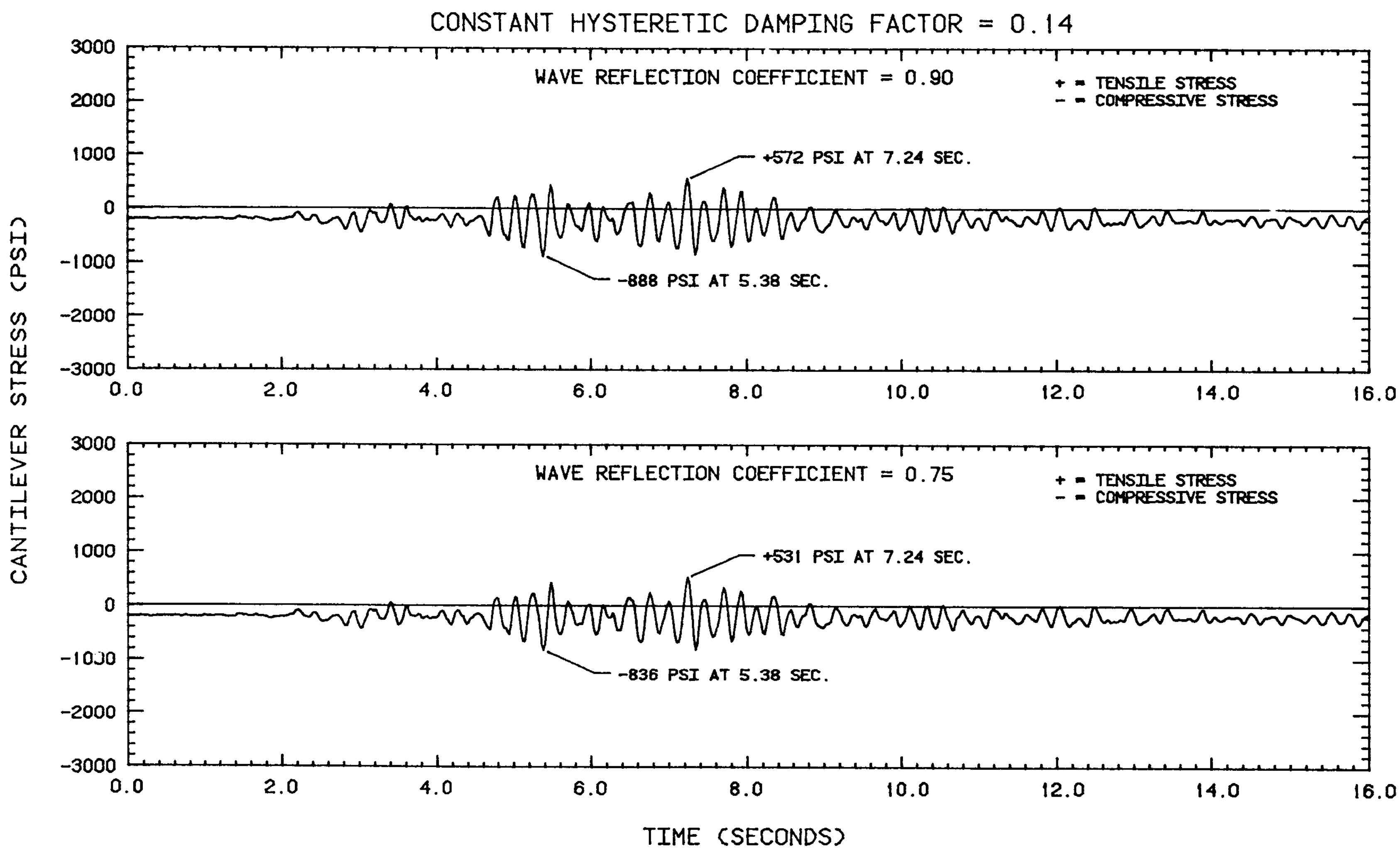


FIGURE E-5 CANTILEVER STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

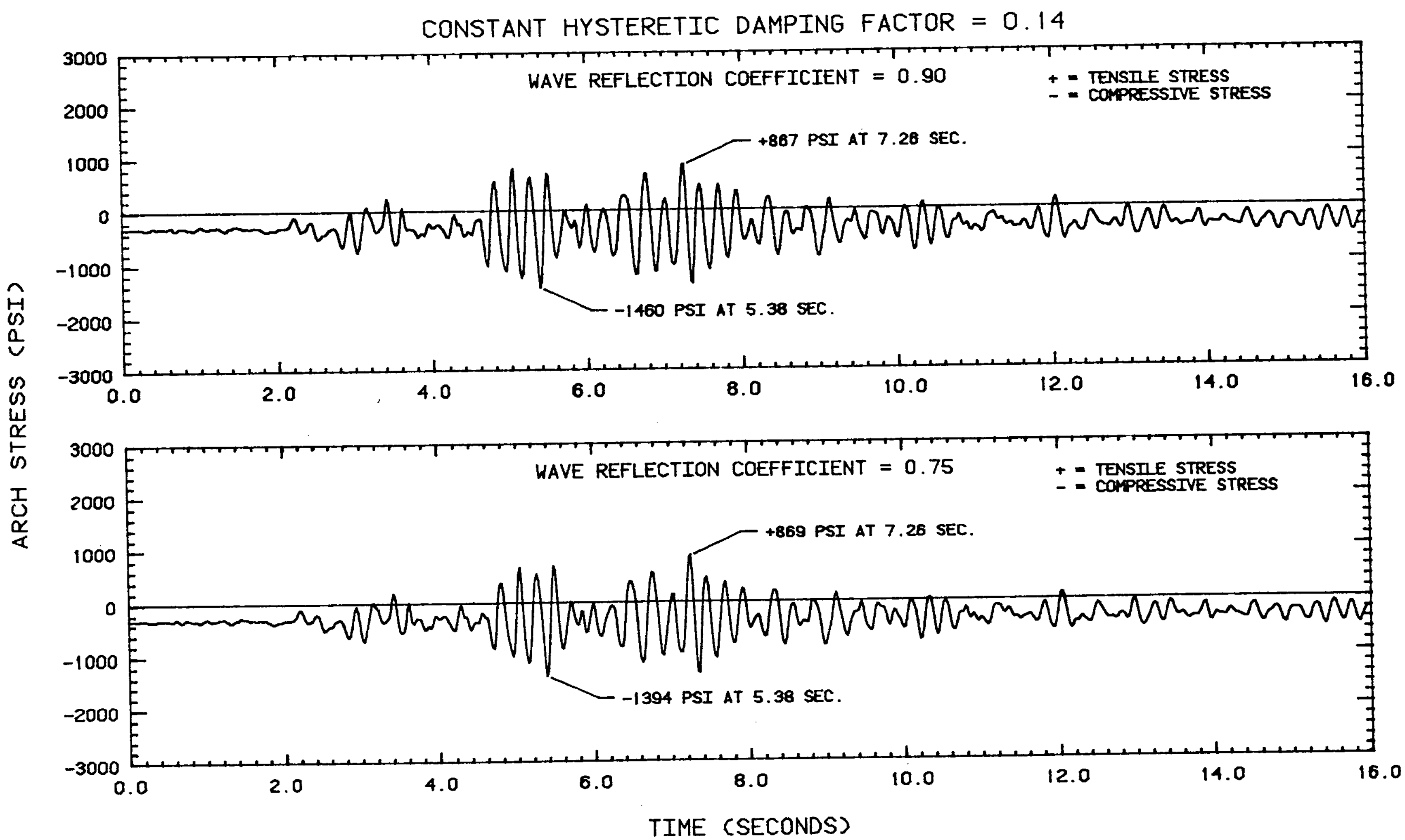


FIGURE E-6 ARCH STRESS RESPONSE AT NODAL POINT 217 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

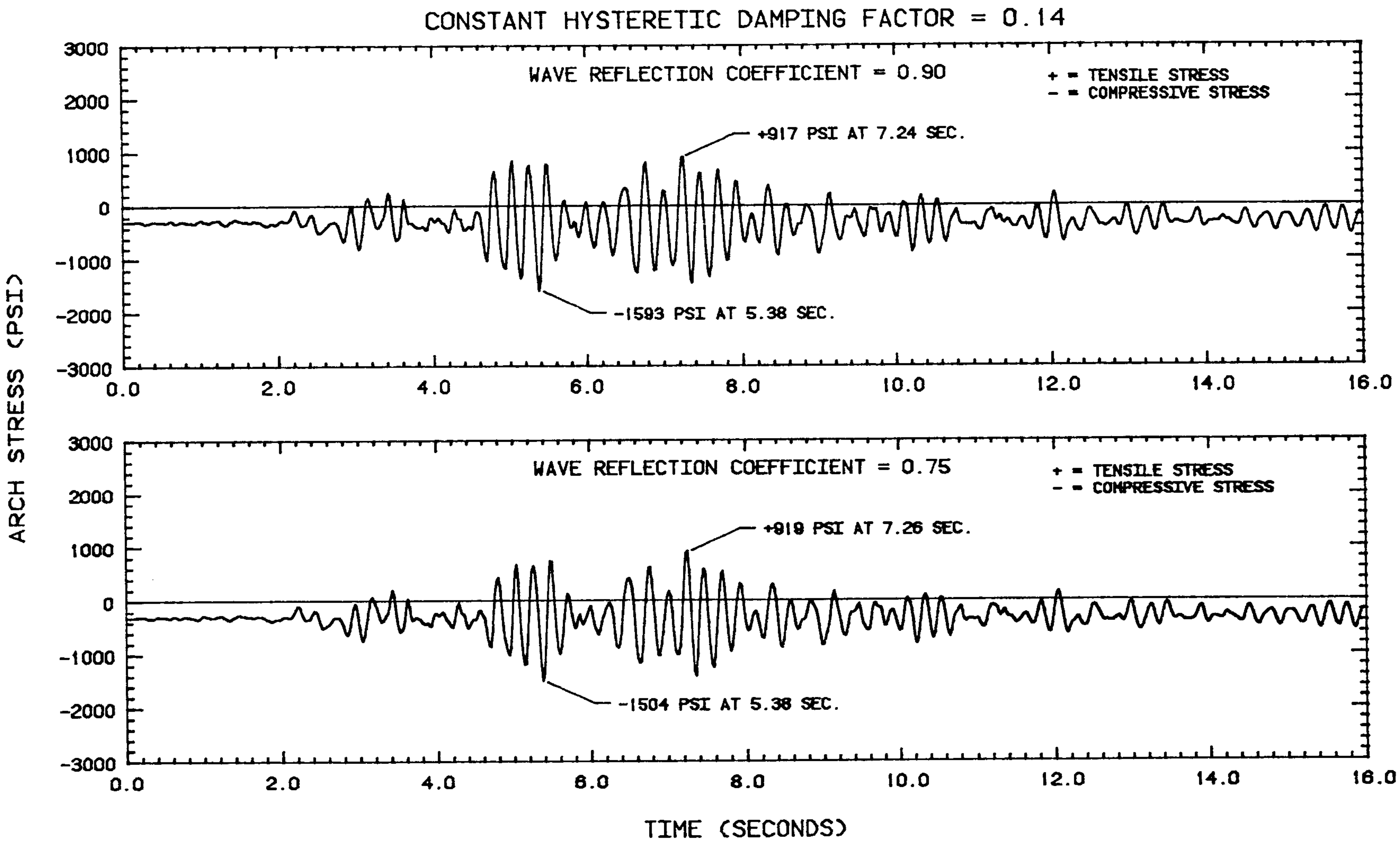


FIGURE E-7 ARCH STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

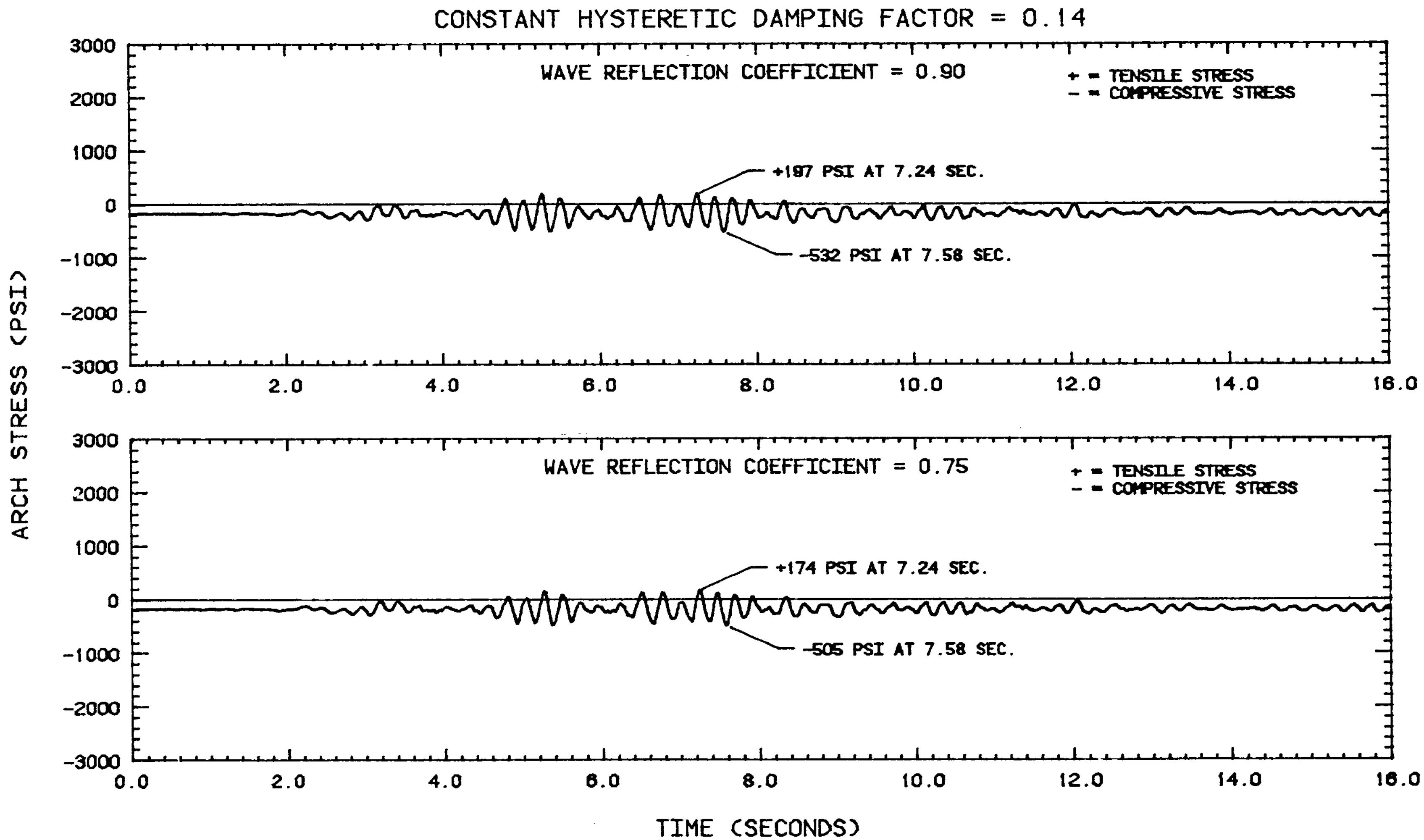


FIGURE E-8 ARCH STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 38 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

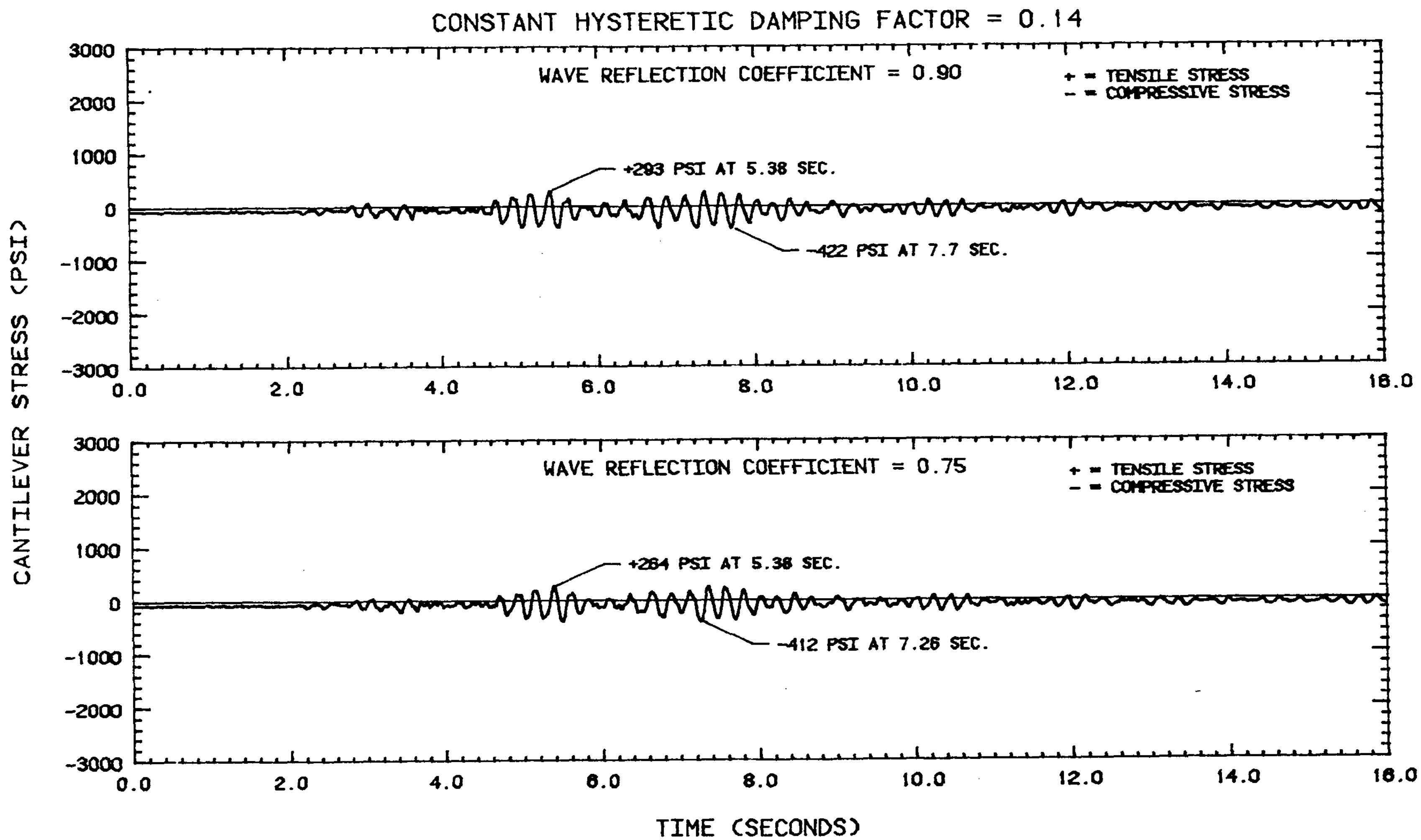


FIGURE E-9 CANTILEVER STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 38 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

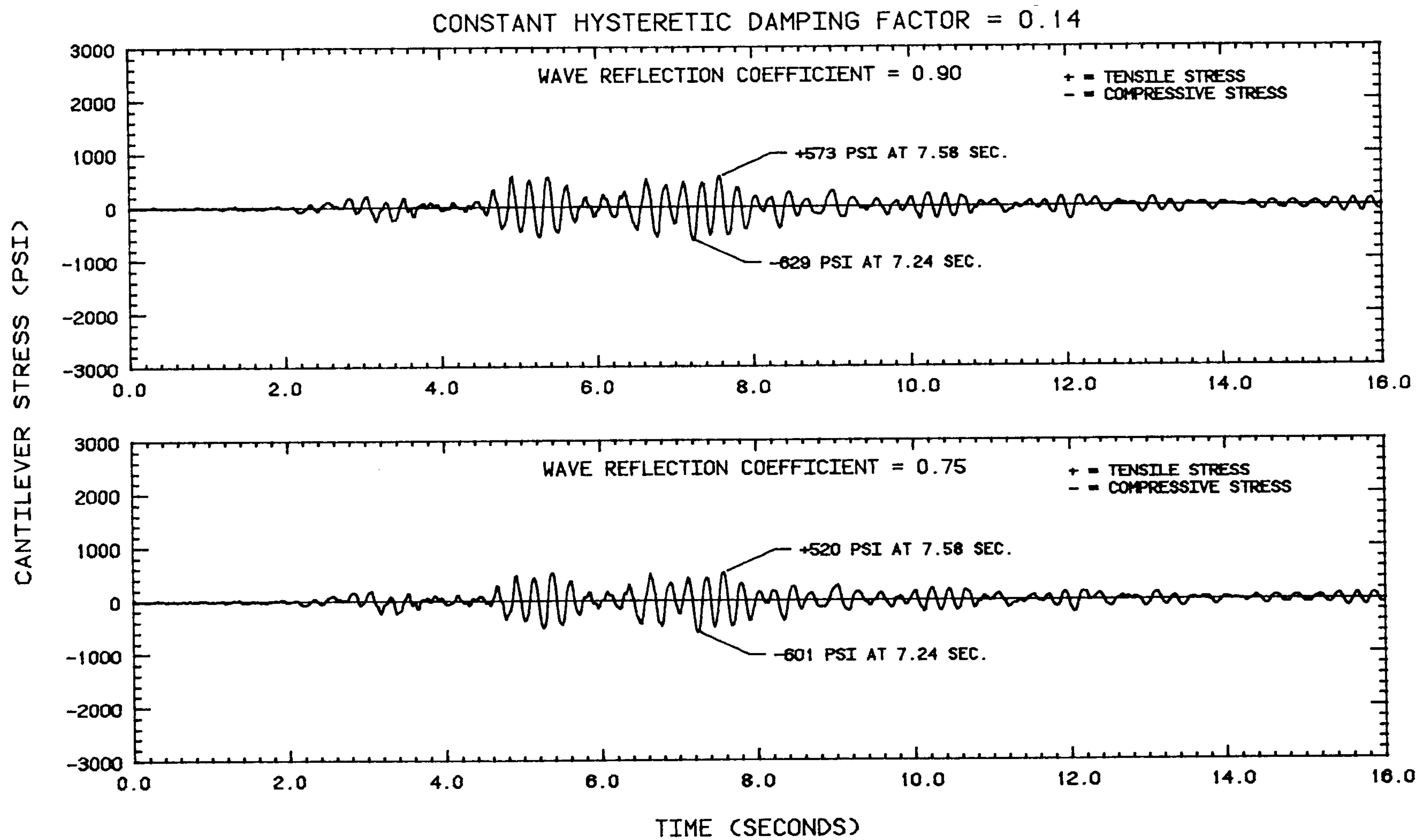


FIGURE E-10 CANTILEVER STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 56 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

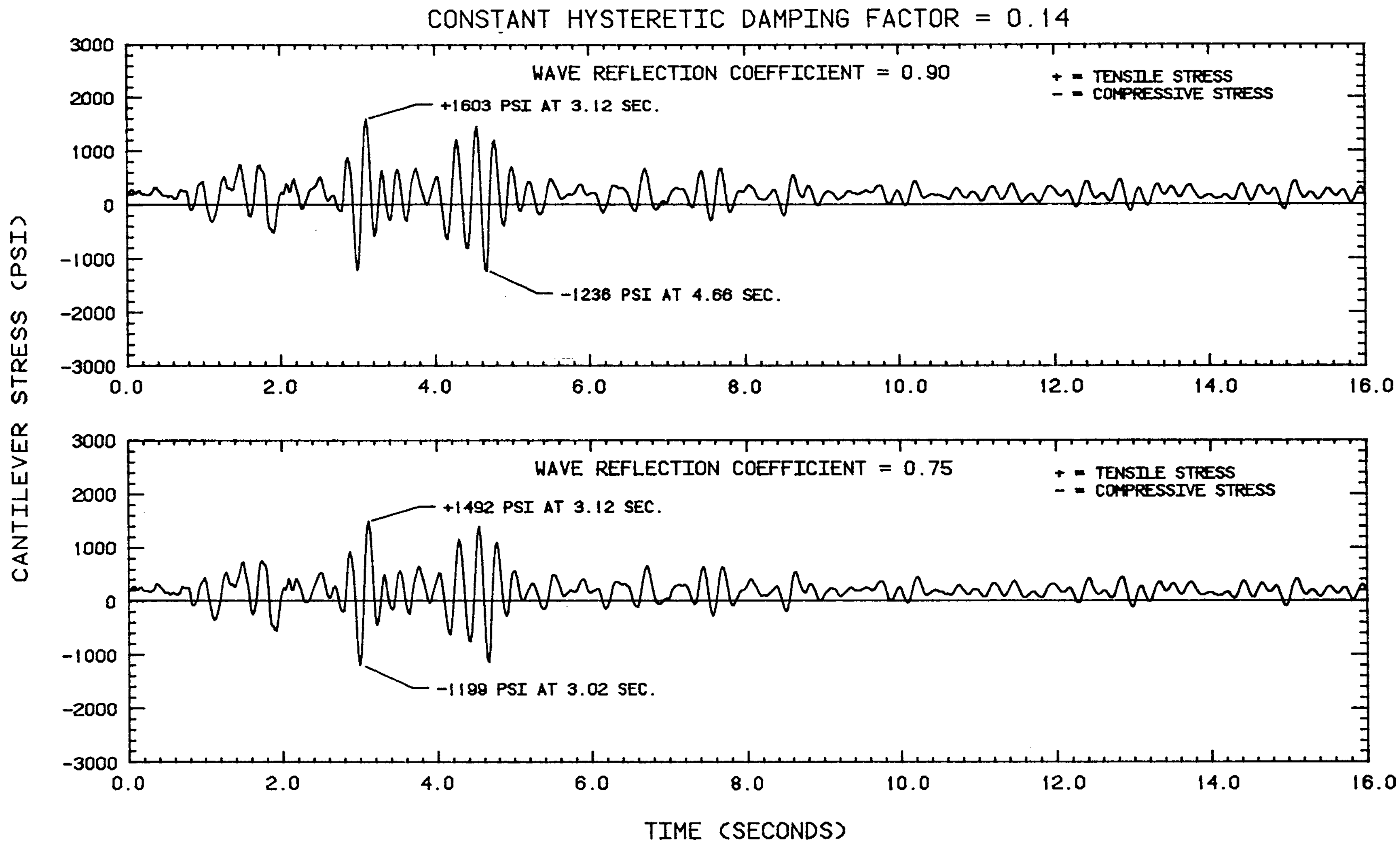


FIGURE E-11 CANTILEVER STRESS RESPONSE AT NODAL POINT 472 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

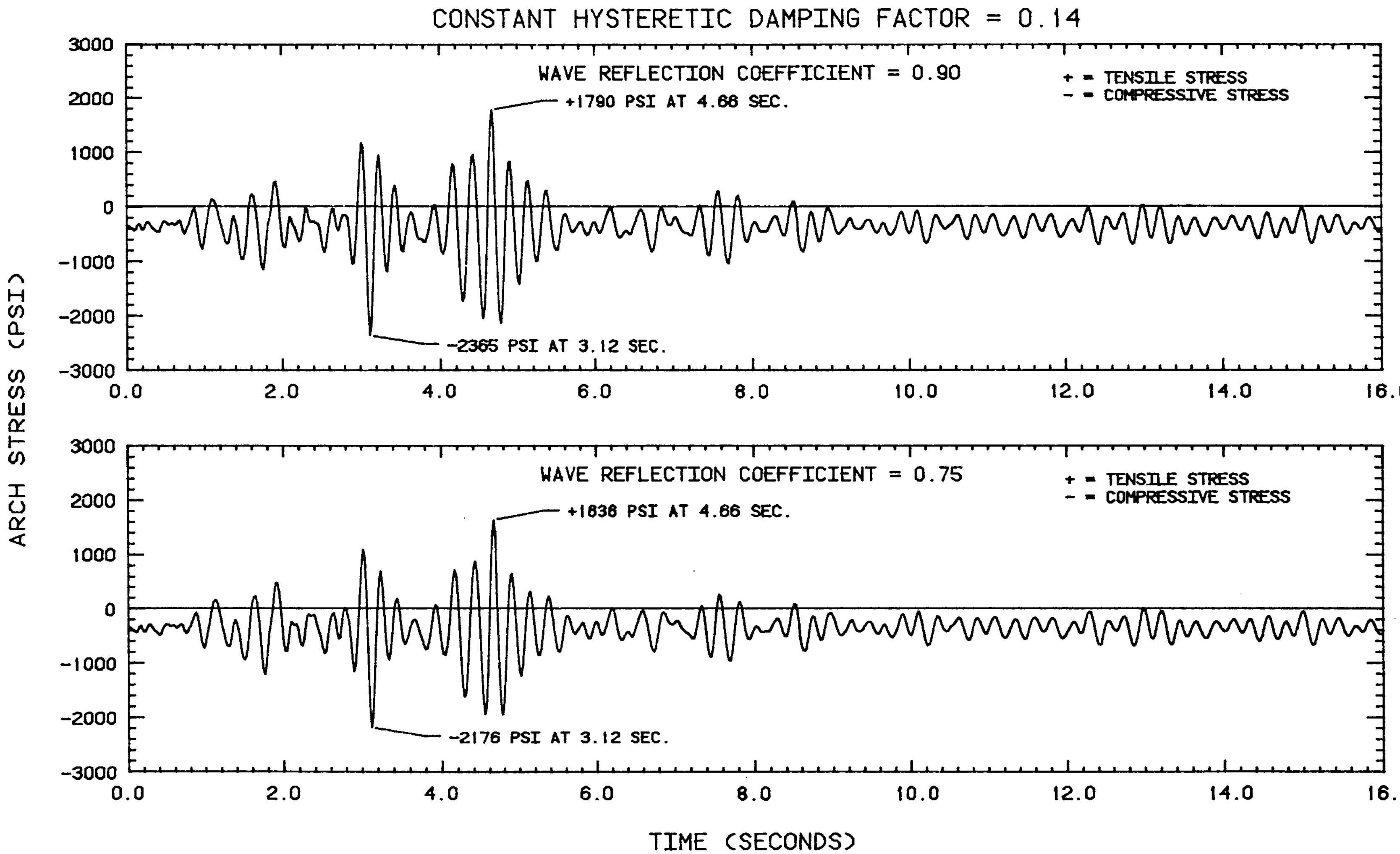


FIGURE E-12 ARCH STRESS RESPONSE AT NODAL POINT 303 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

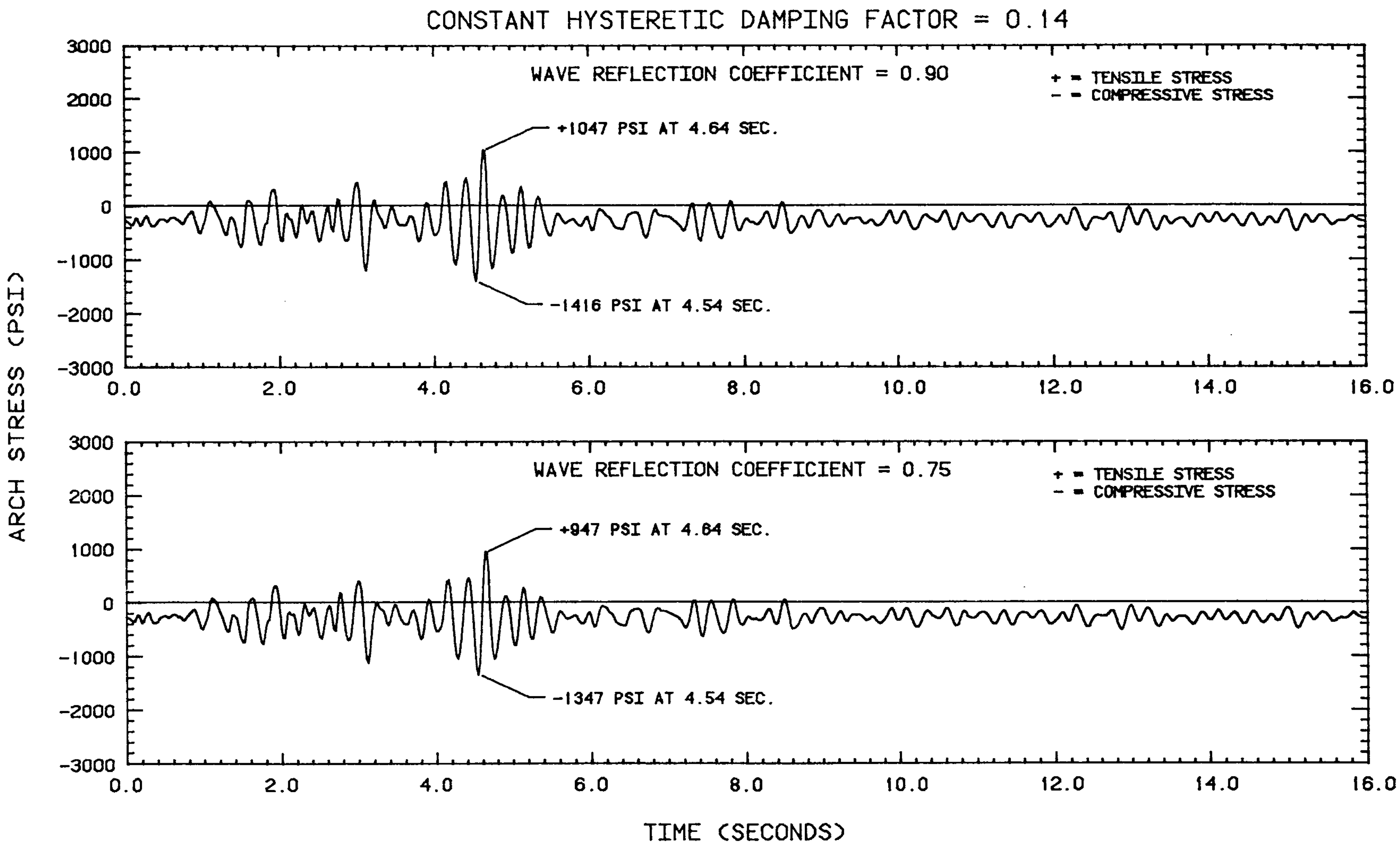


FIGURE E-13 ARCH STRESS RESPONSE AT NODAL POINT 22 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

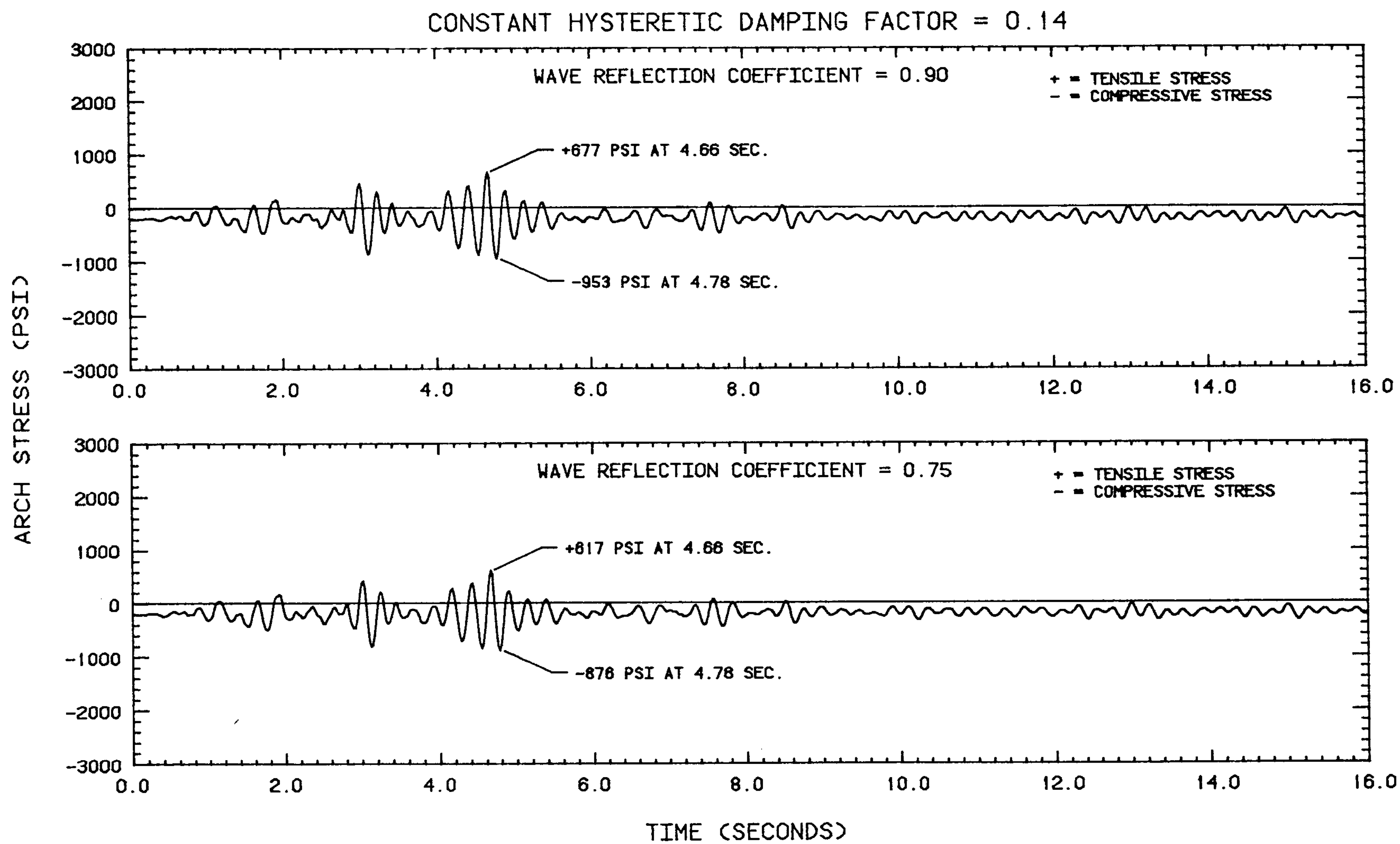


FIGURE E-14 ARCH STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

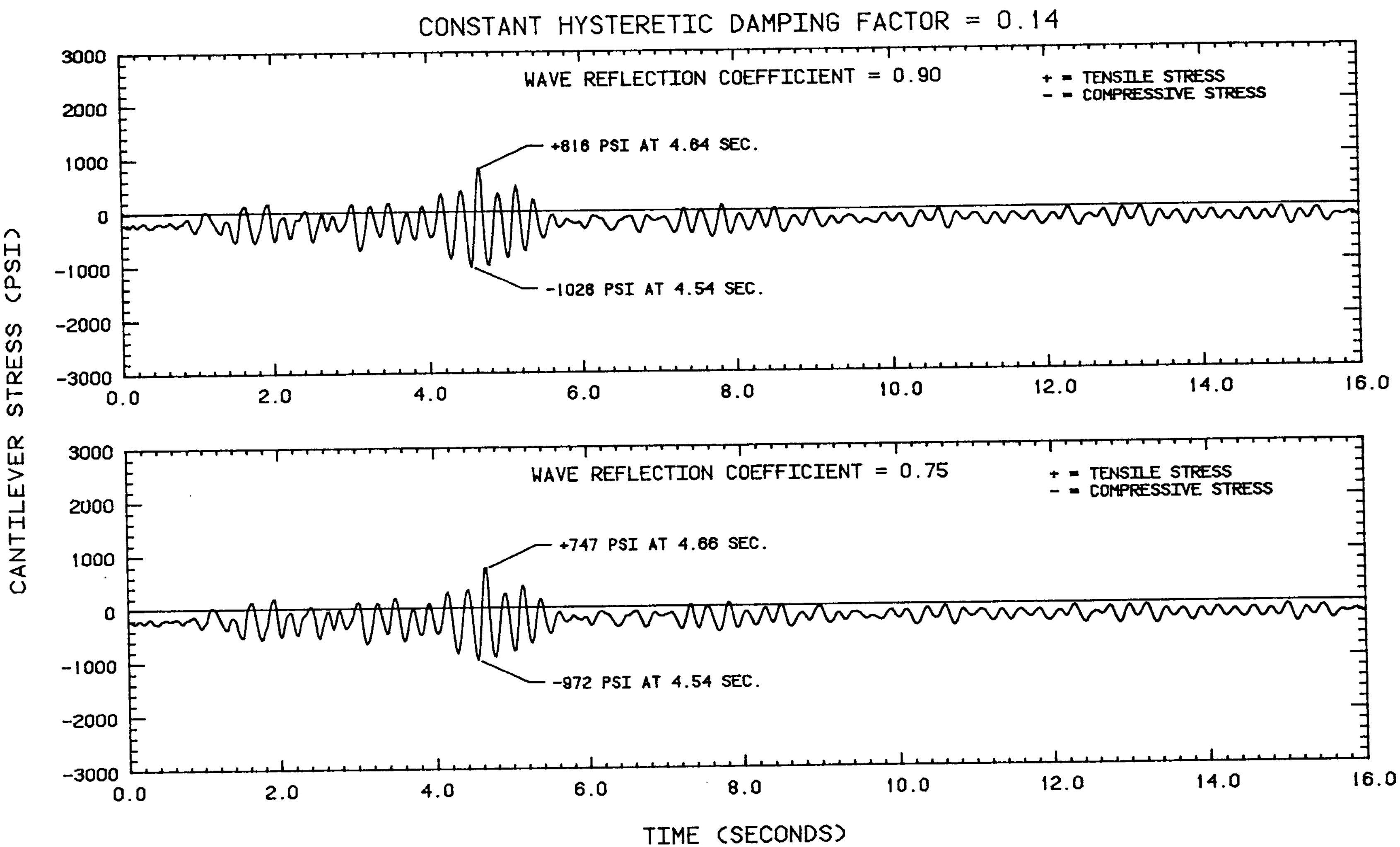


FIGURE E-15 CANTILEVER STRESS RESPONSE AT NODAL POINT 149 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

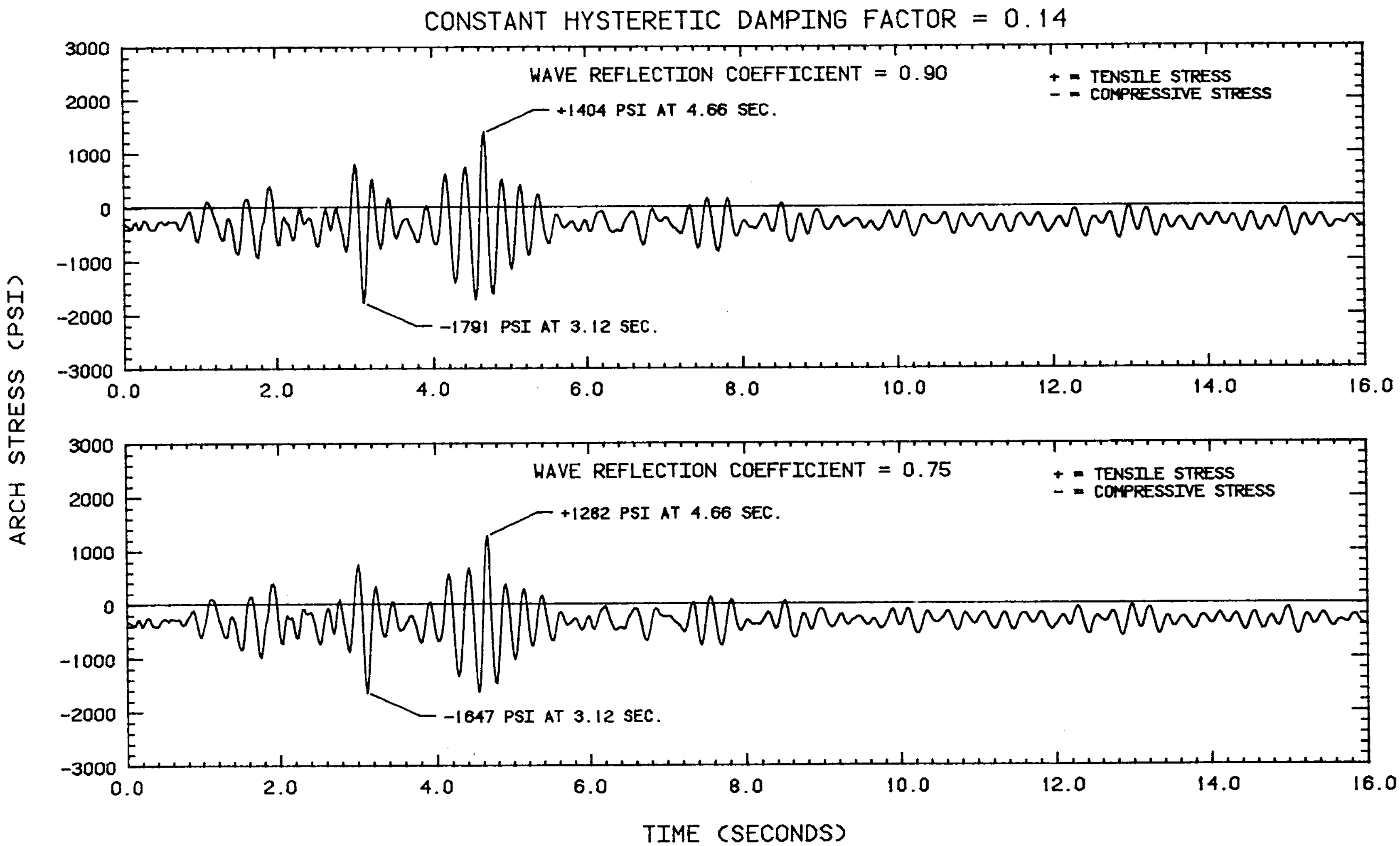


FIGURE E-16 ARCH STRESS RESPONSE AT NODAL POINT 217 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

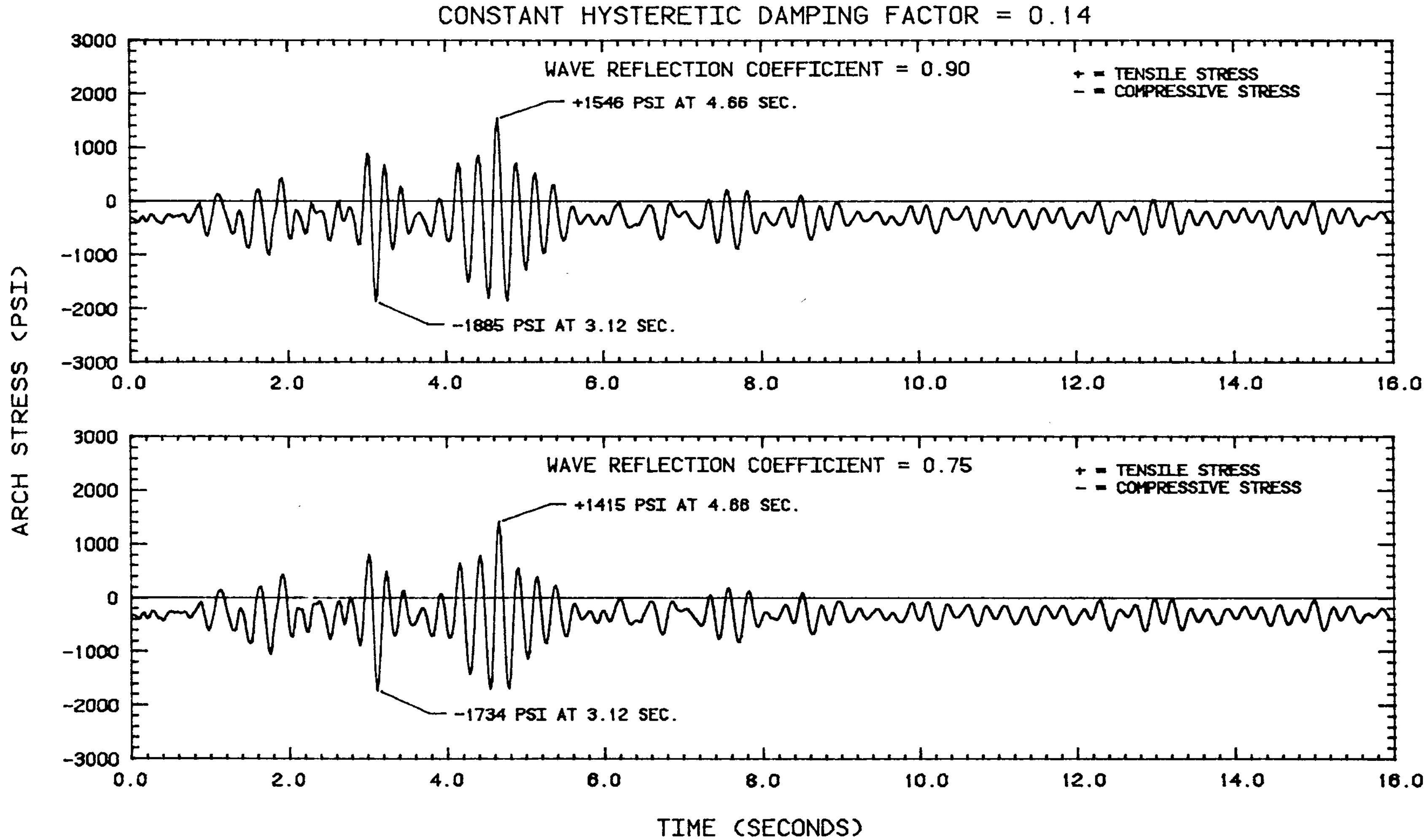


FIGURE E-17 ARCH STRESS RESPONSE AT STRESS LOCATION 4 IN ELEMENT 9 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

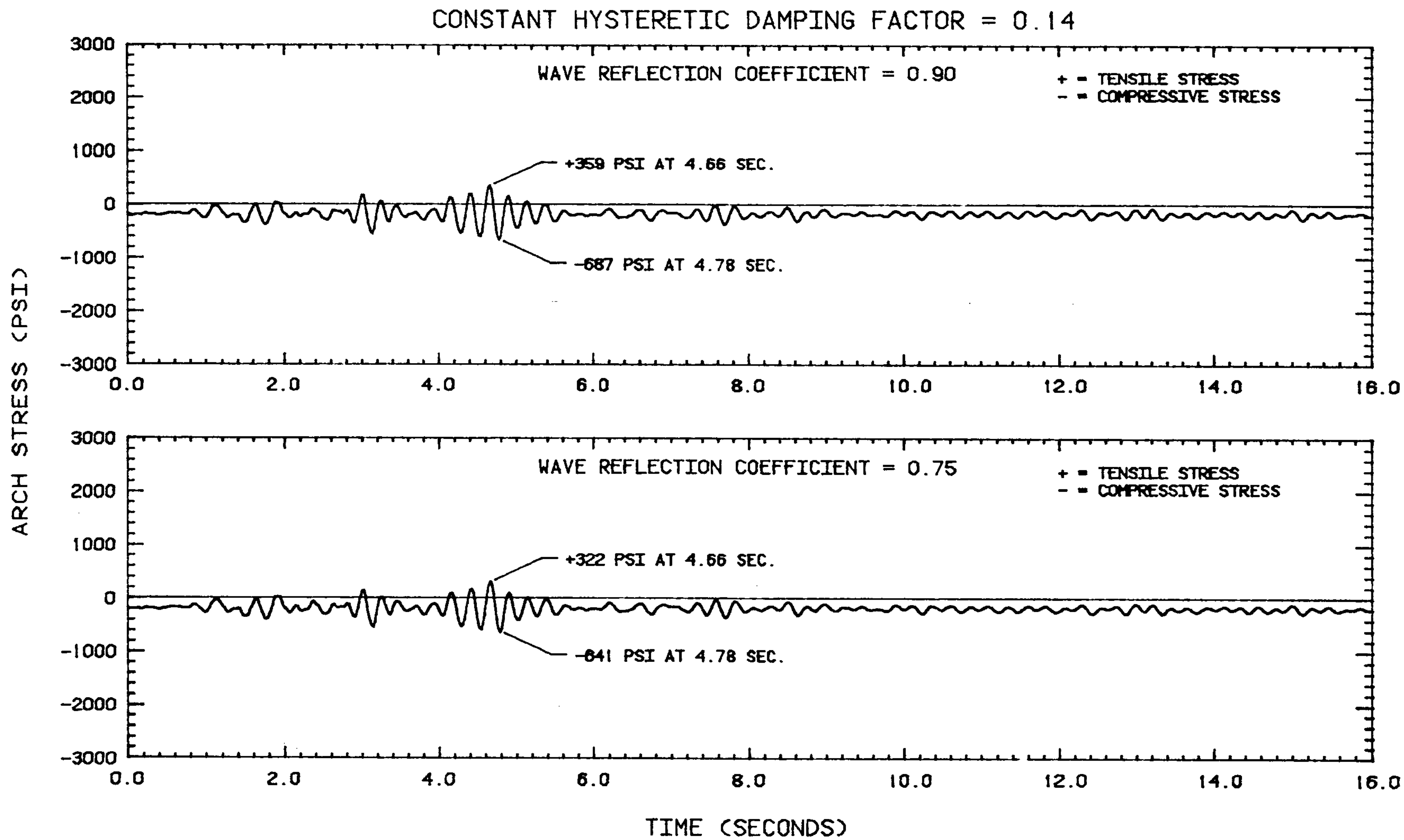


FIGURE E-18 ARCH STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 38 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

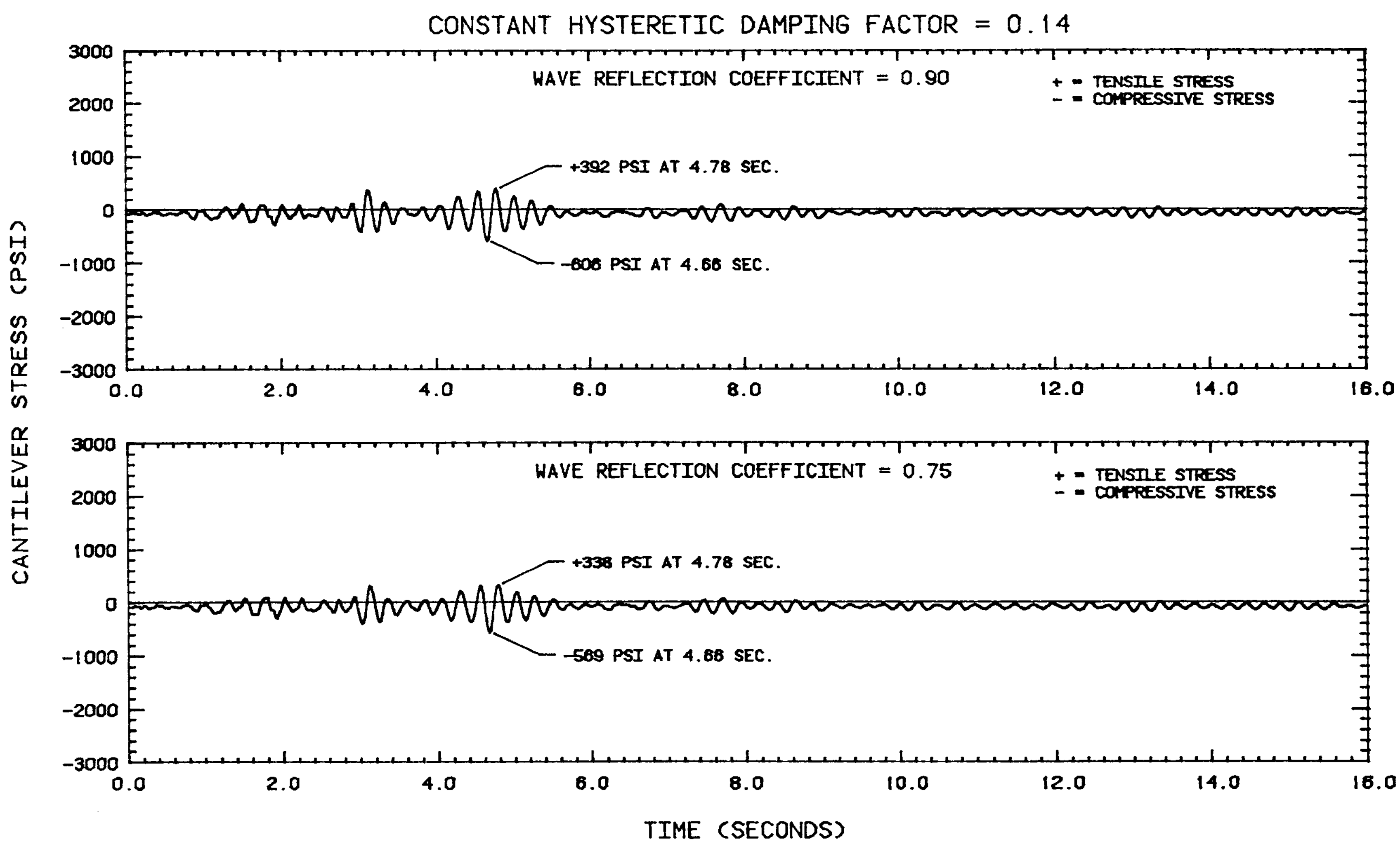


FIGURE E-19 CANTILEVER STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 38 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

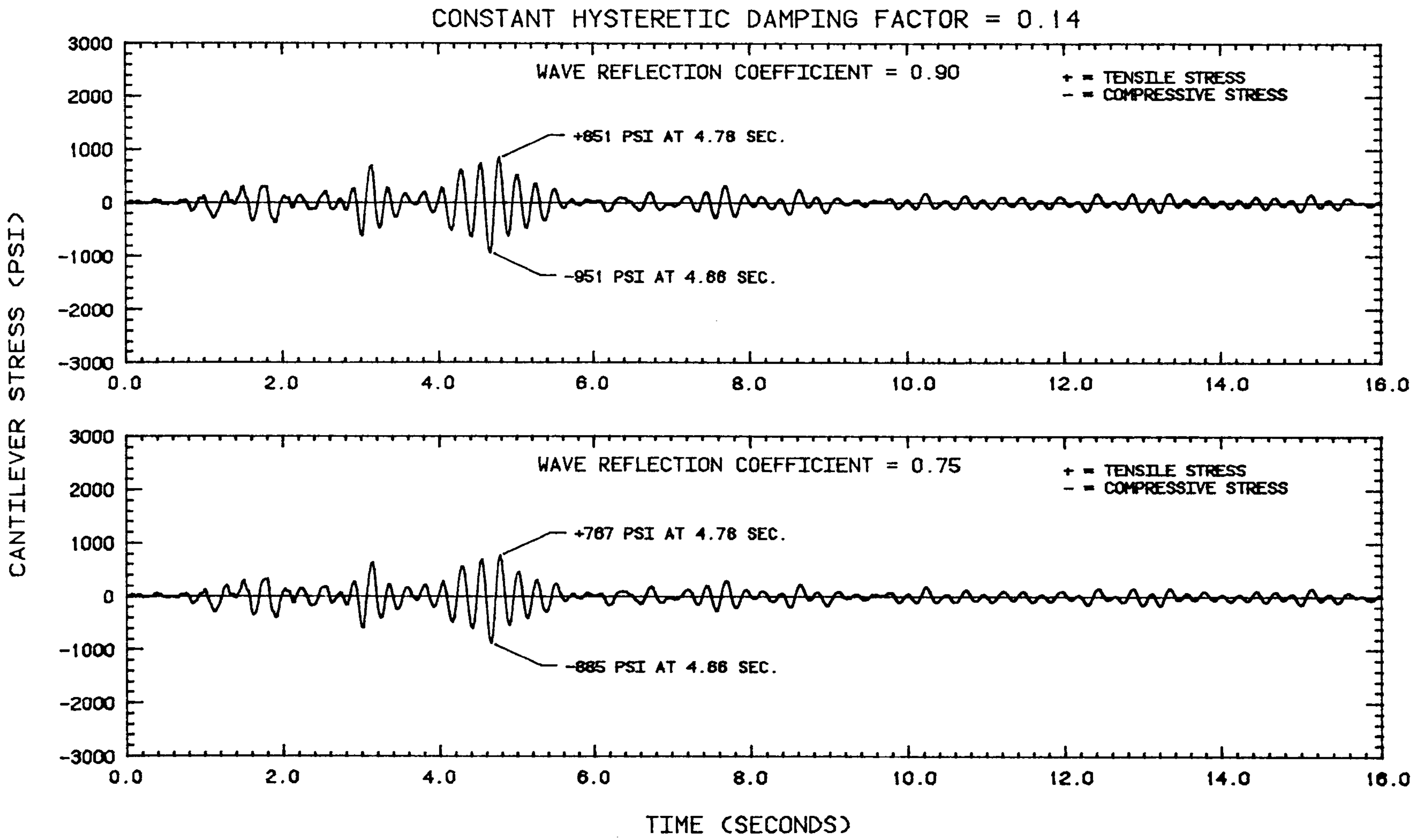


FIGURE E-20 CANTILEVER STRESS RESPONSE AT STRESS LOCATION 2 IN ELEMENT 56 DUE TO QUAKE 2, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).

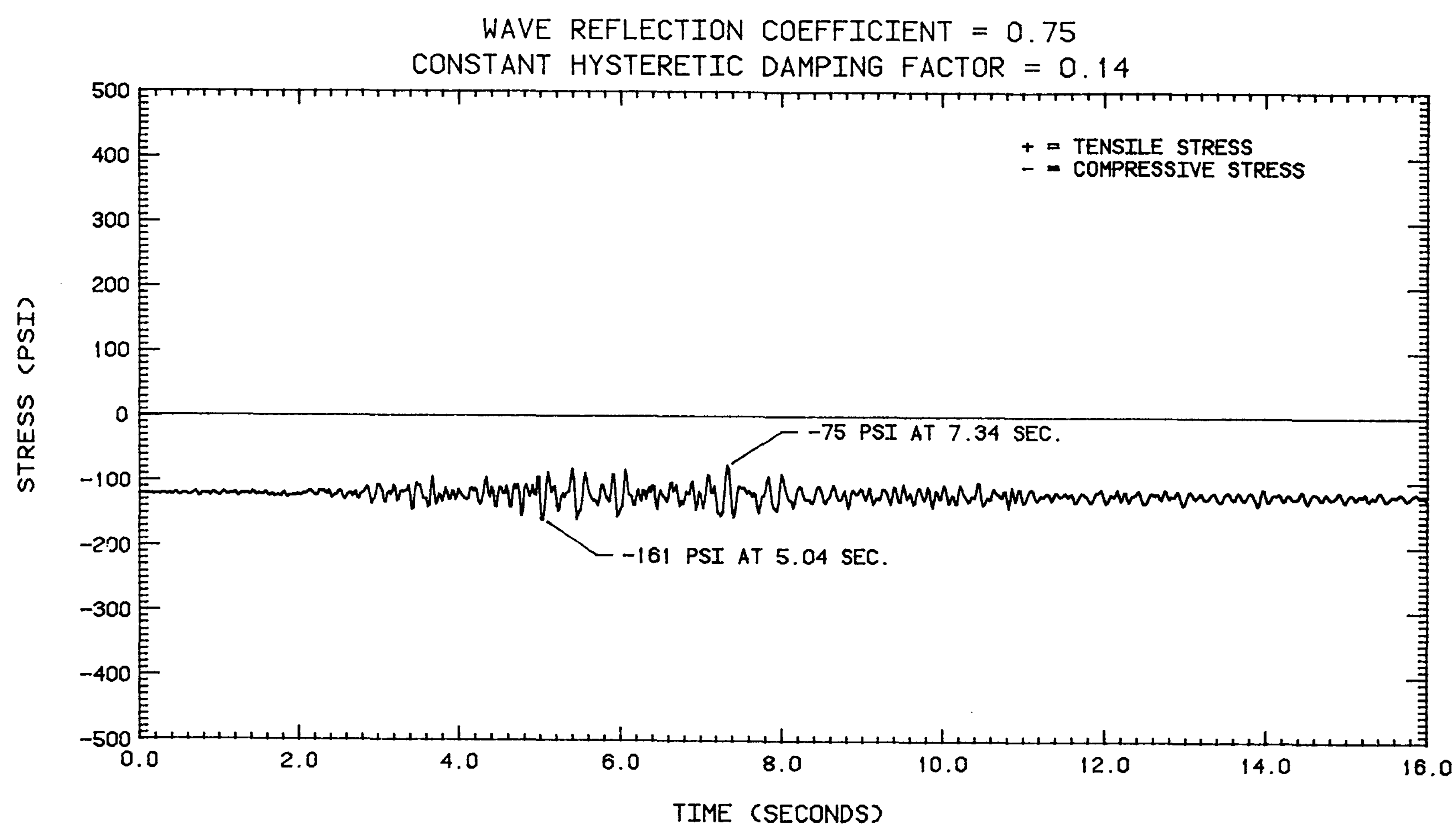


FIGURE E-21 ARCH STRESS RESPONSE AT NODAL POINT 474 DUE TO QUAKE 1, INCLUDING THE STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE. RESPONSE IS COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET).
NOTE: STRESS TIME-HISTORY LIES ENTIRELY IN COMPRESSION DOMAIN.

STRESSES ALONG SECTION 6

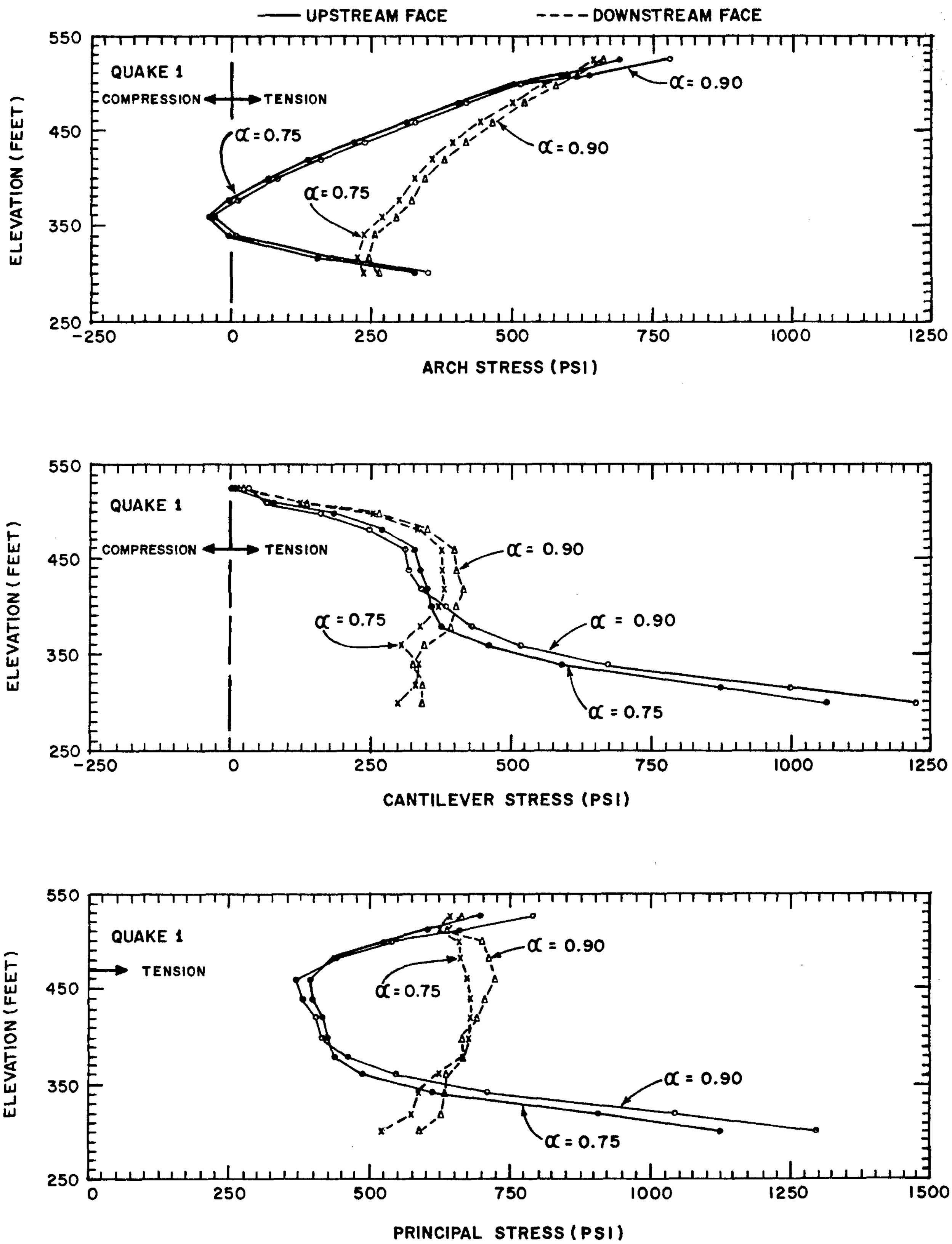


FIGURE E-22 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG SECTION 6. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

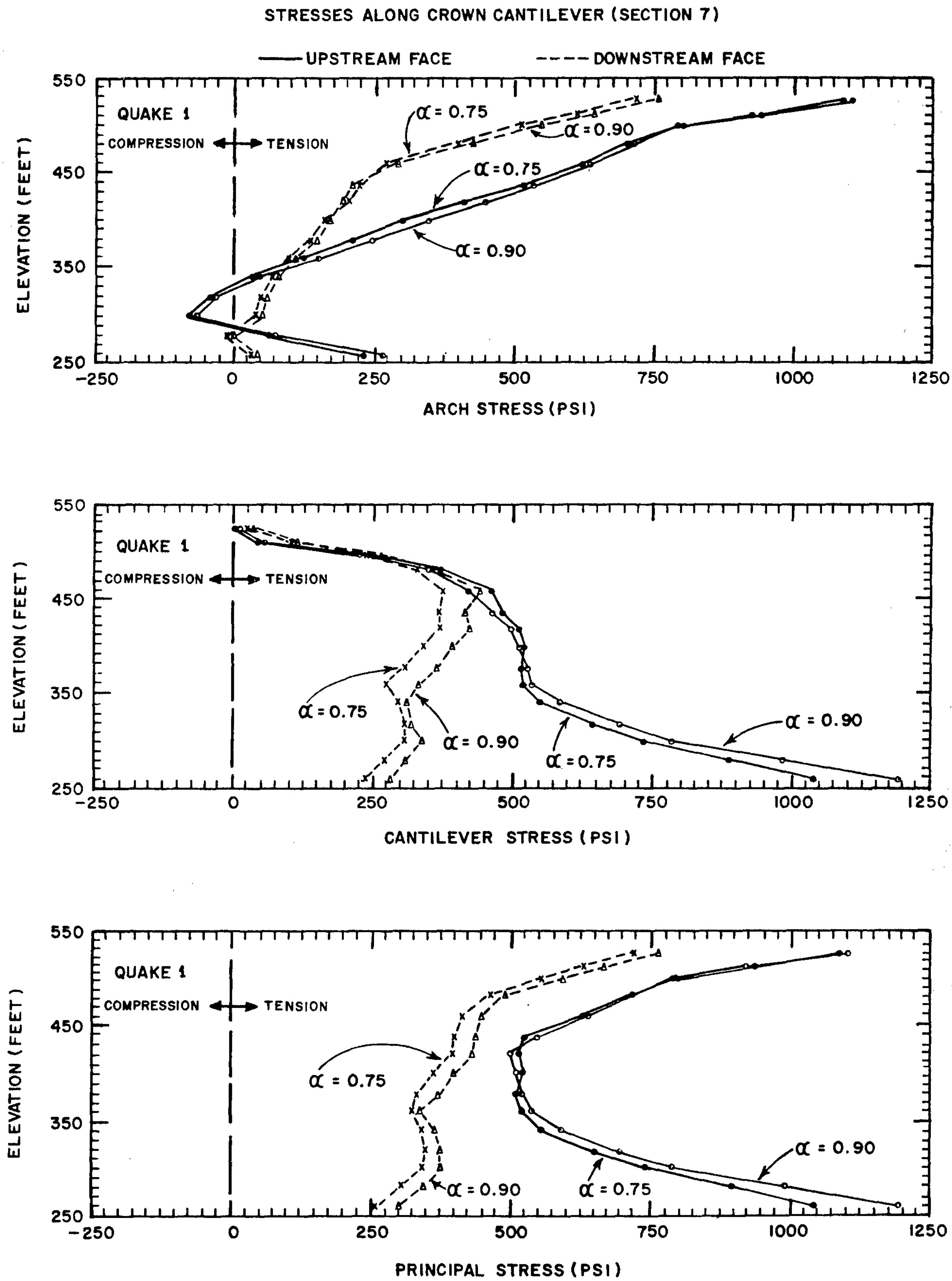


FIGURE E-23 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG CROWN CANTILEVER. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ALONG SECTION 8

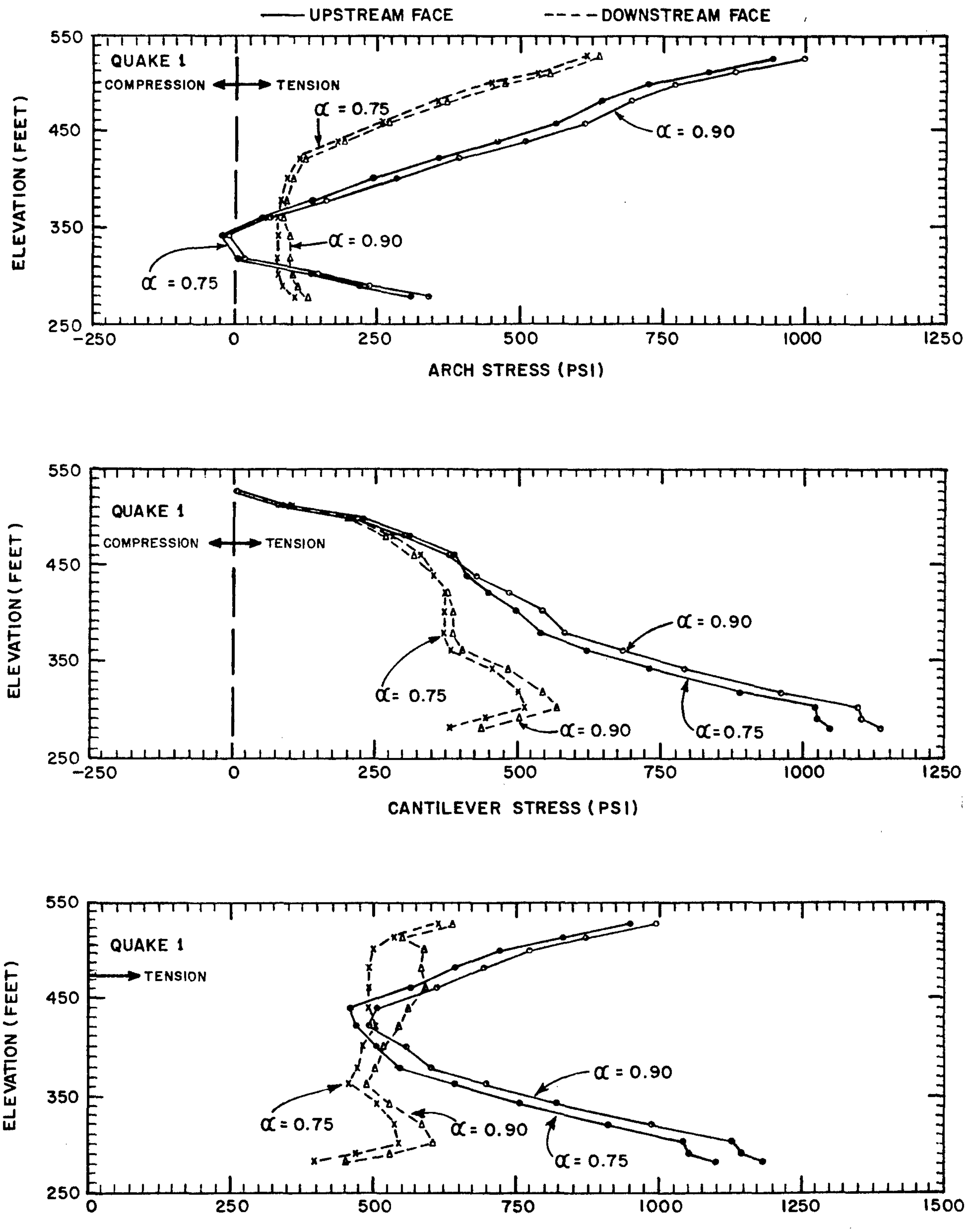


FIGURE E-24 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG SECTION 8. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

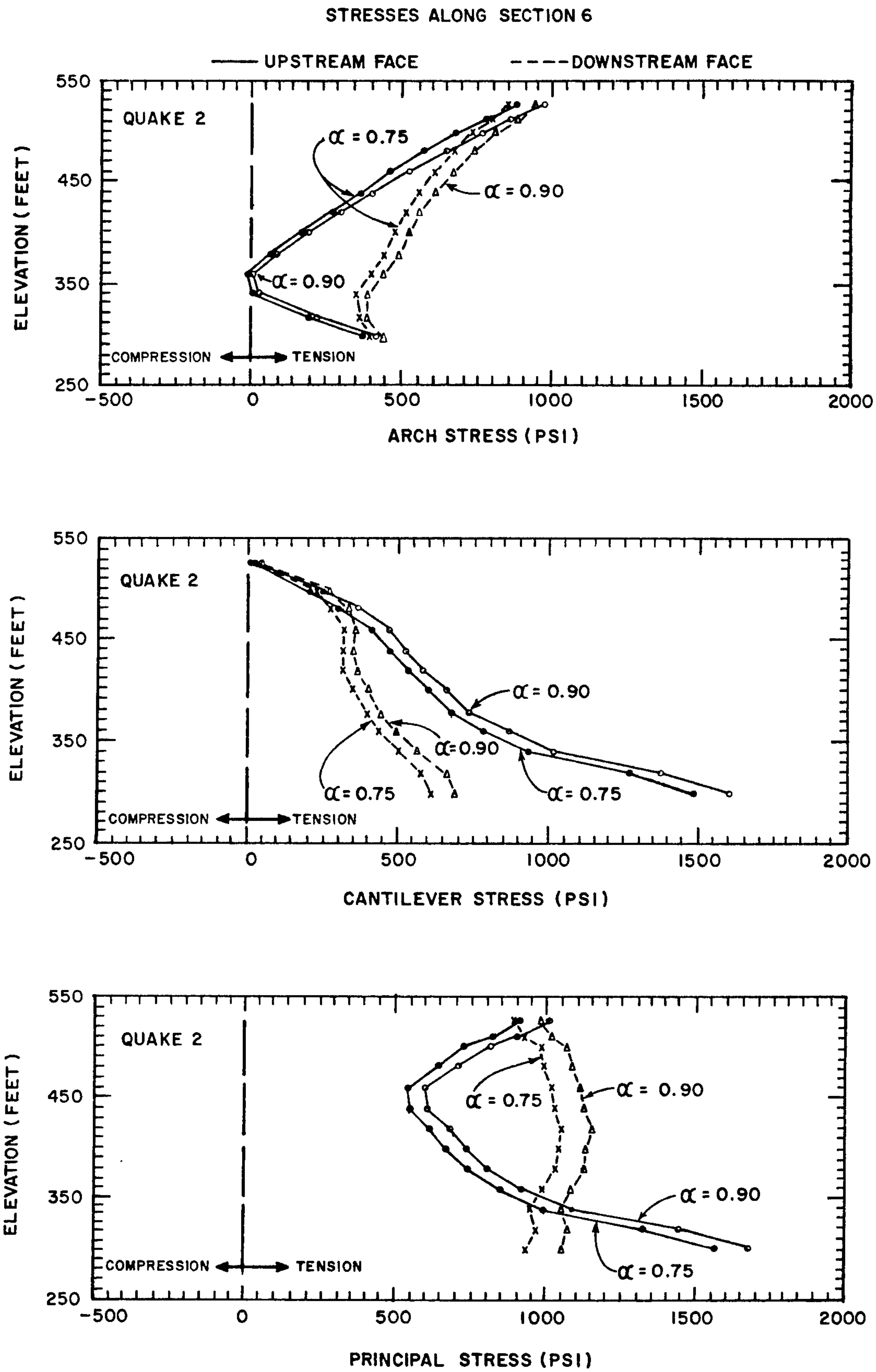


FIGURE E-25 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG SECTION 6. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

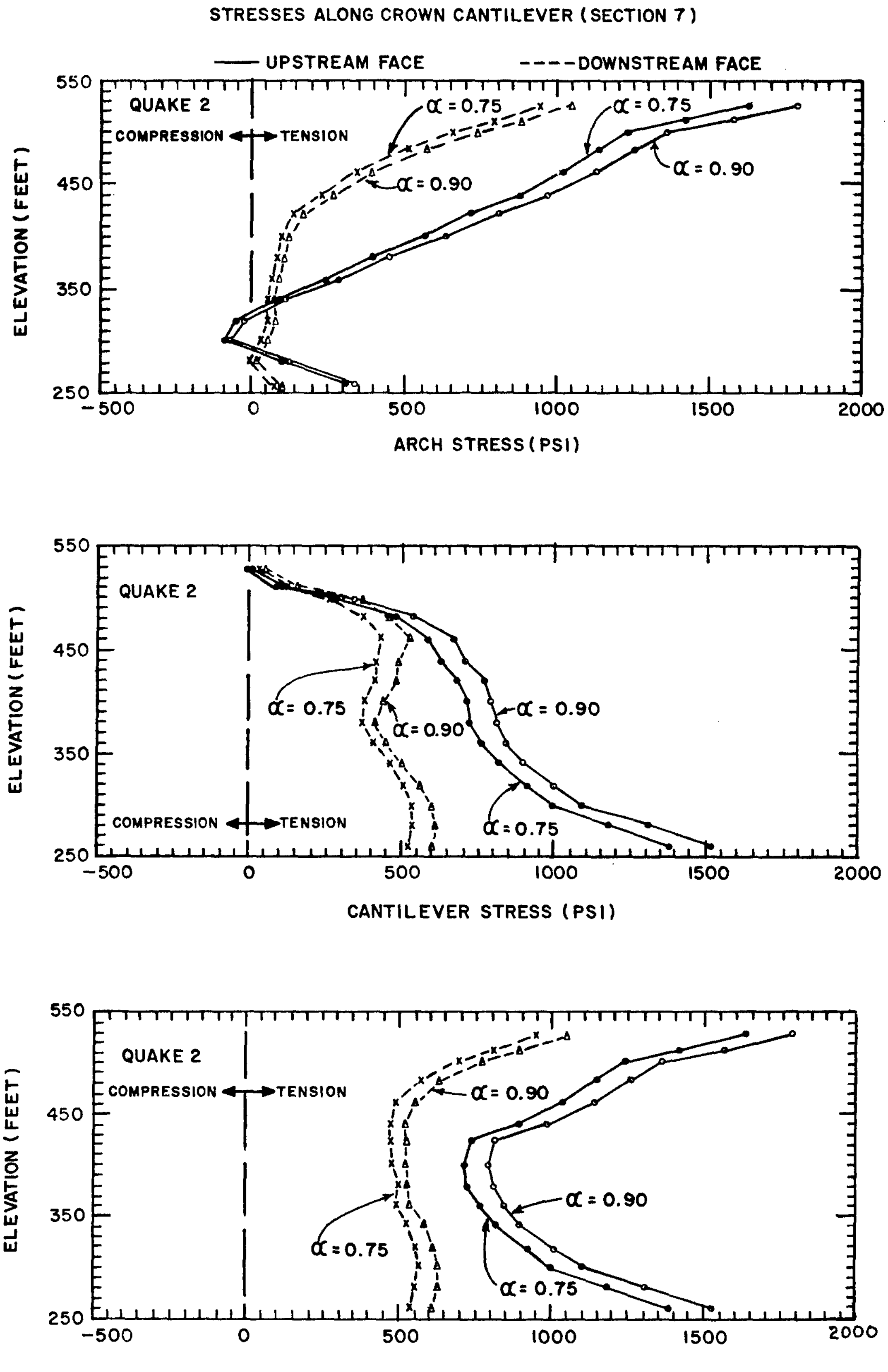


FIGURE E-26 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG CROWN CANTILEVER. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

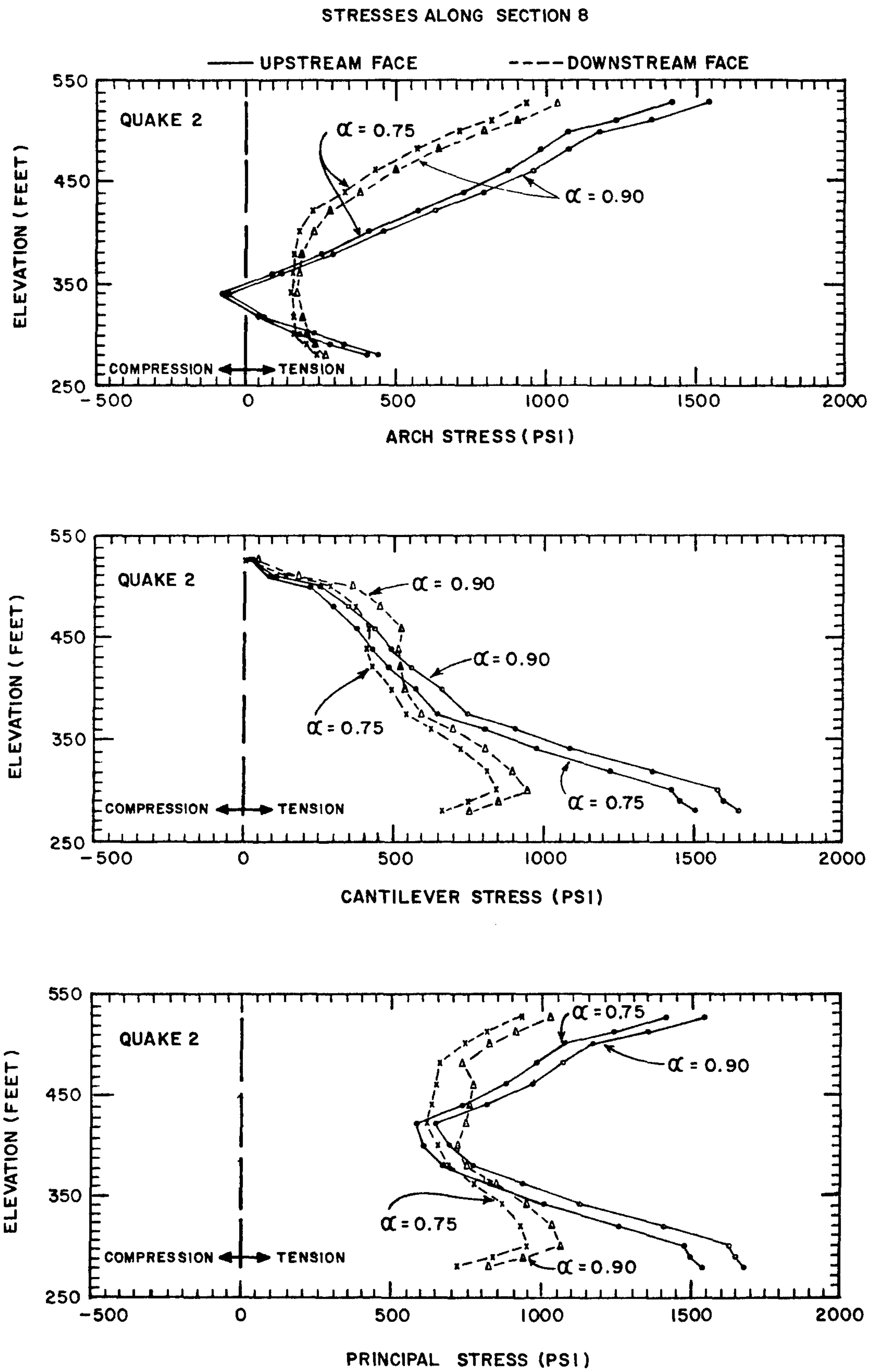


FIGURE E-27 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON UPSTREAM AND DOWNSTREAM FACES OF THE DAM ALONG SECTION 8. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

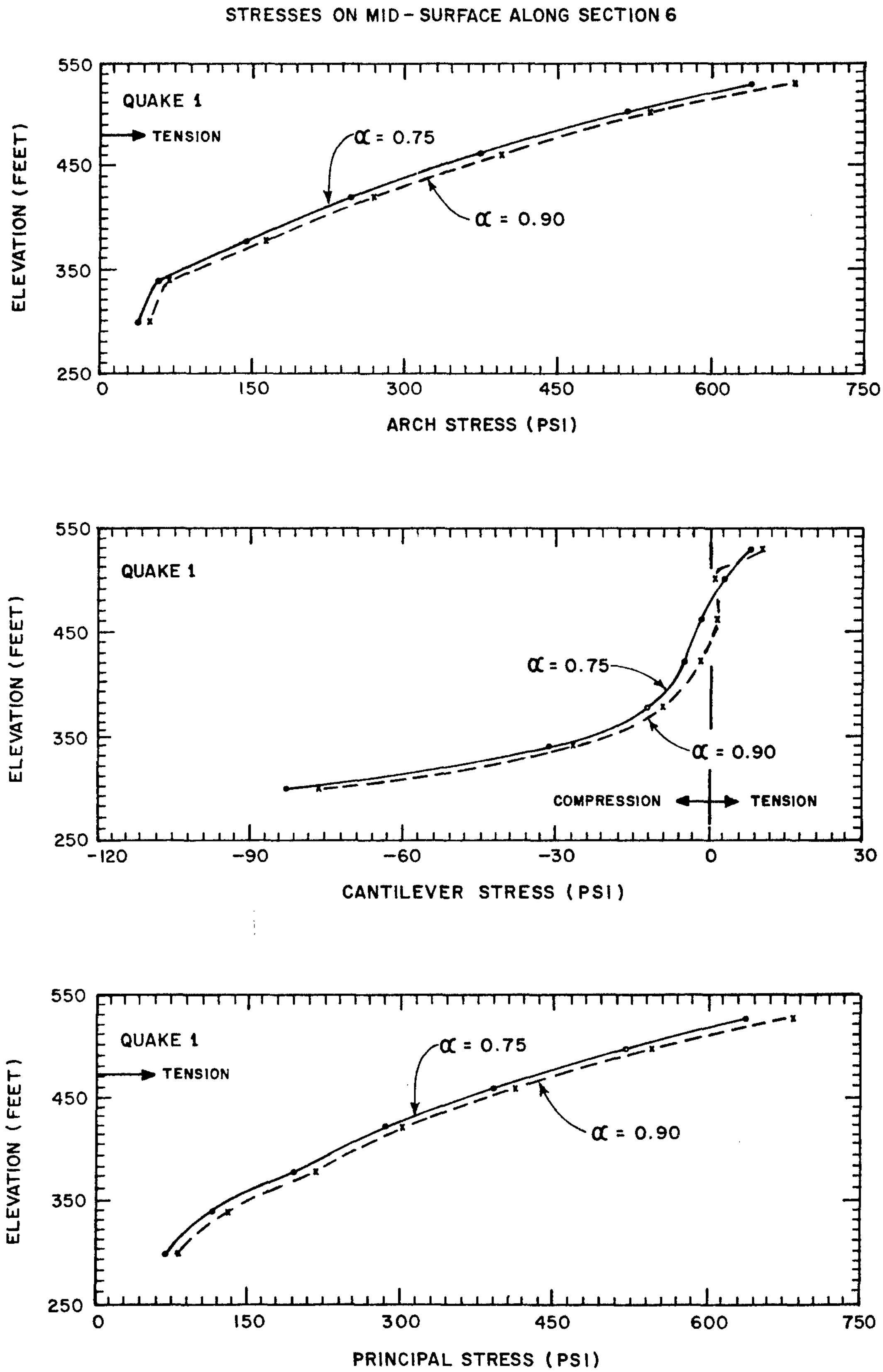
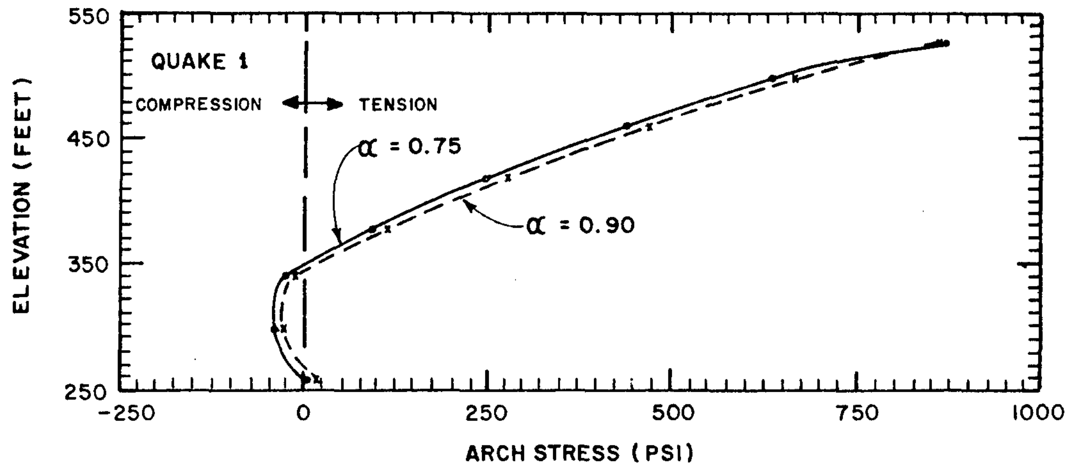
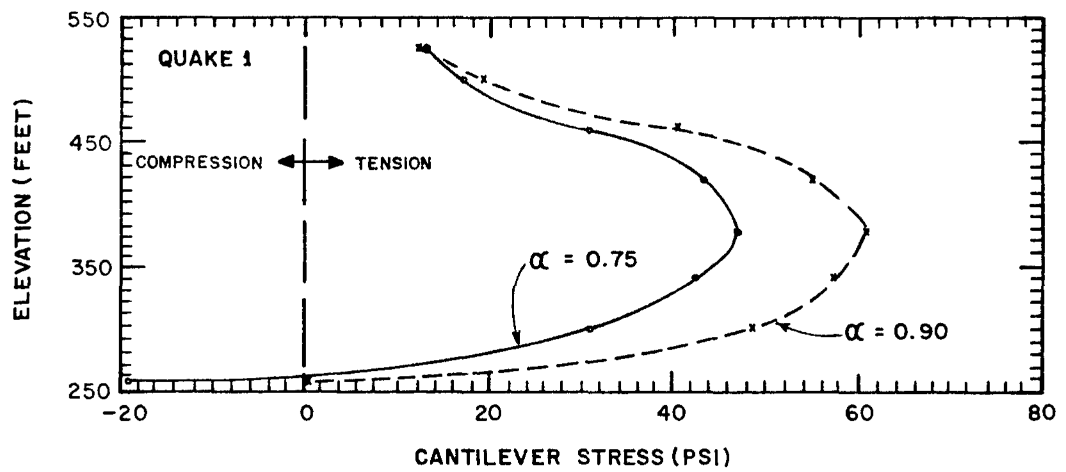


FIGURE E-28 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID-SURFACE OF THE DAM ALONG SECTION 6. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ON MID-SURFACE ALONG CROWN CANTILEVER (SECTION 7)





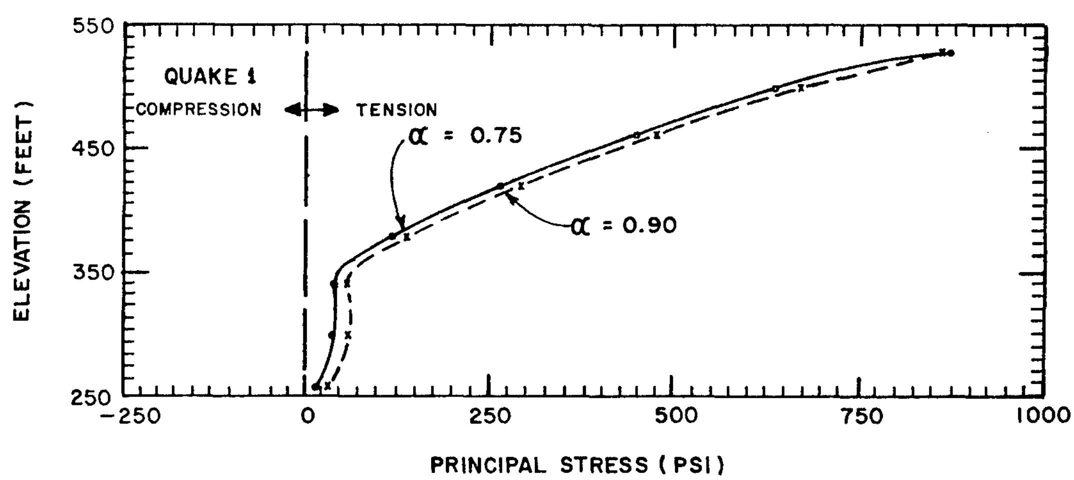


FIGURE E-29 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID-SURFACE OF THE DAM ALONG CROWN CANTILEVER. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ON MID-SURFACE ALONG SECTION 8

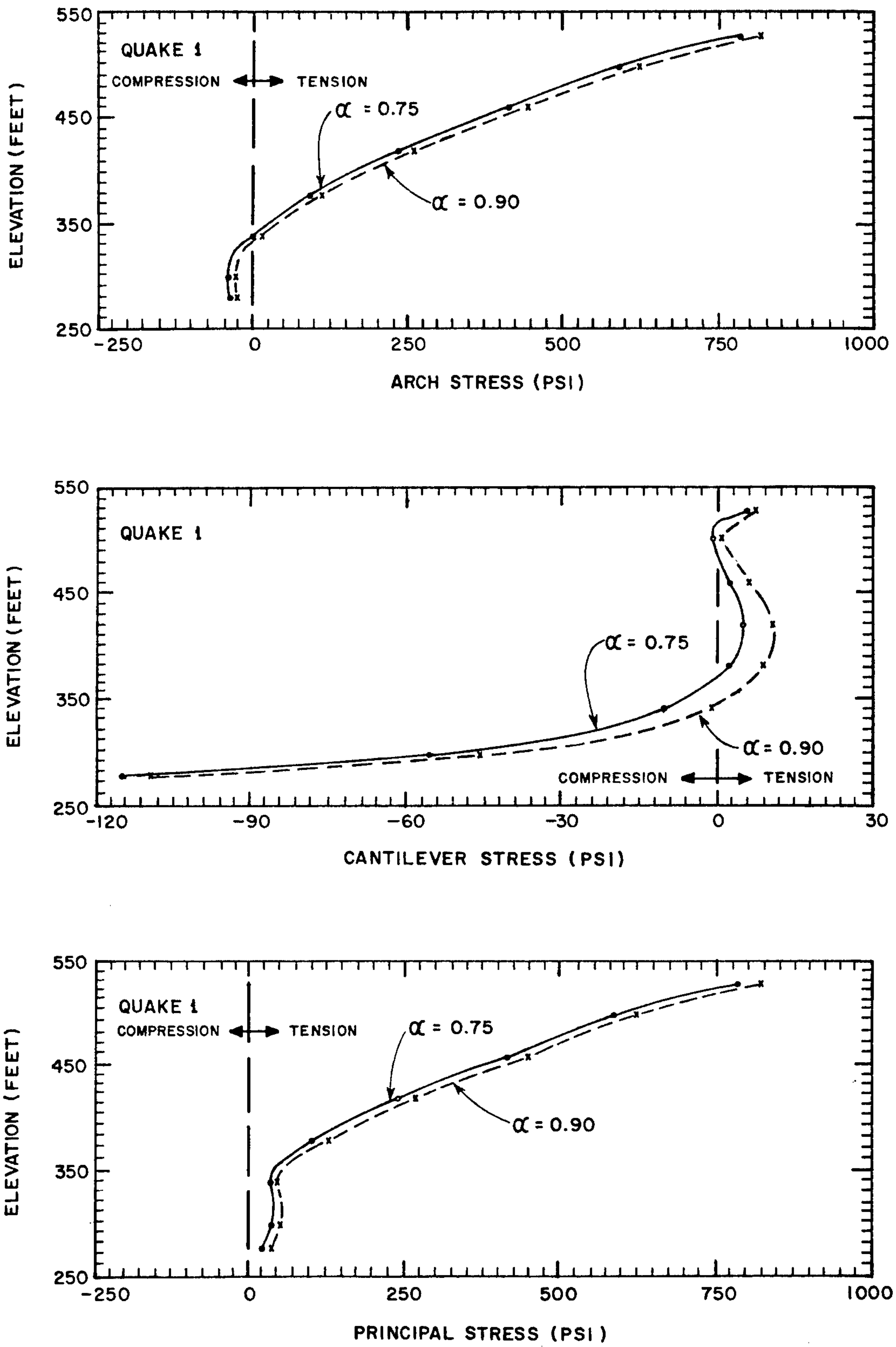
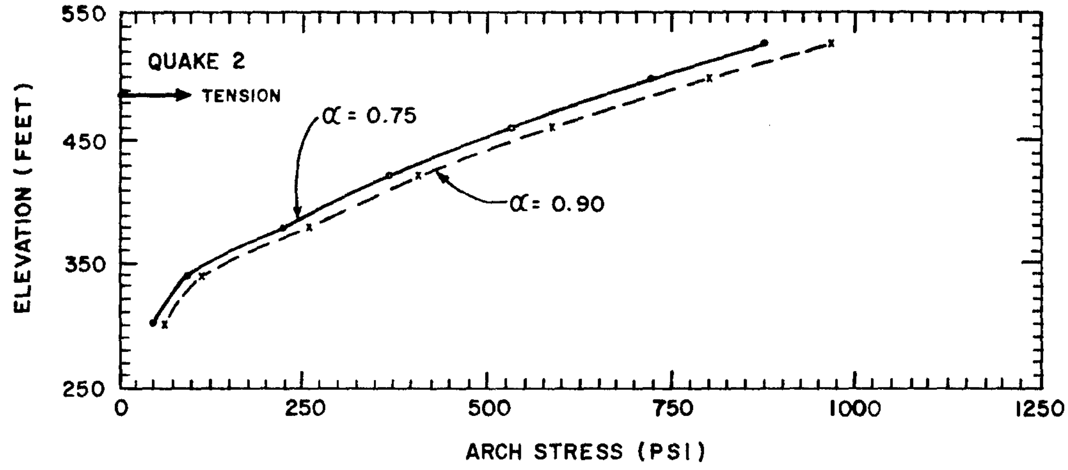
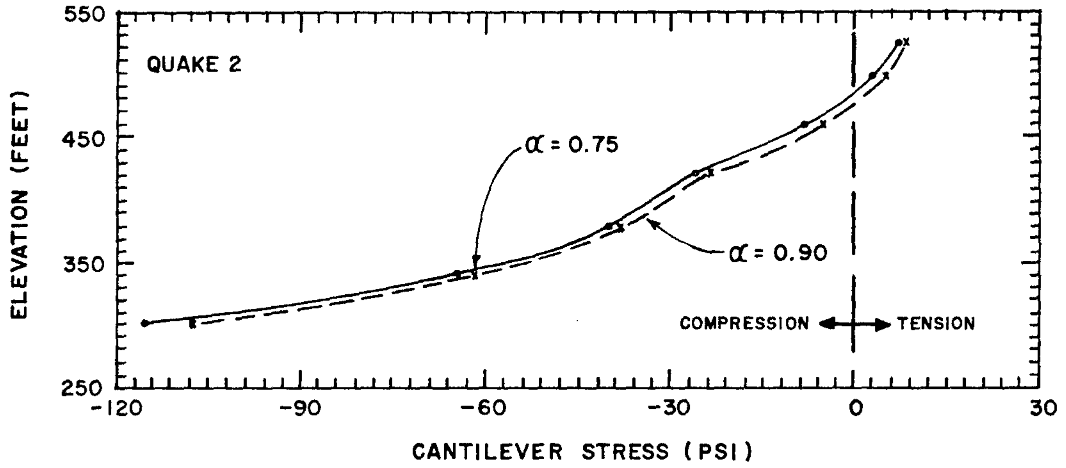


FIGURE E-30 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID -SURFACE OF THE DAM ALONG SECTION 8. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE I EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ON MID-SURFACE ALONG SECTION 6





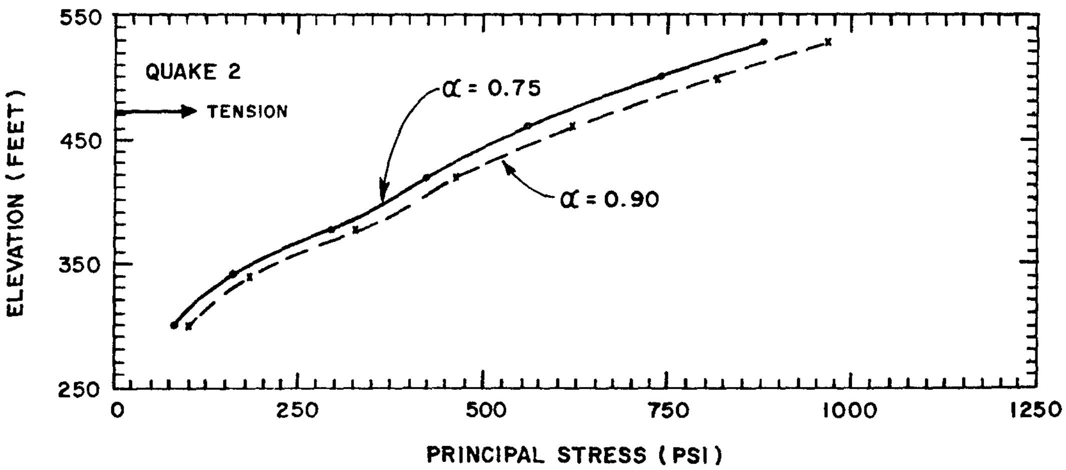


FIGURE E-31 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID-SURFACE OF THE DAM ALONG SECTION 6. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ON MID-SURFACE ALONG CROWN CANTILEVER (SECTION 7)

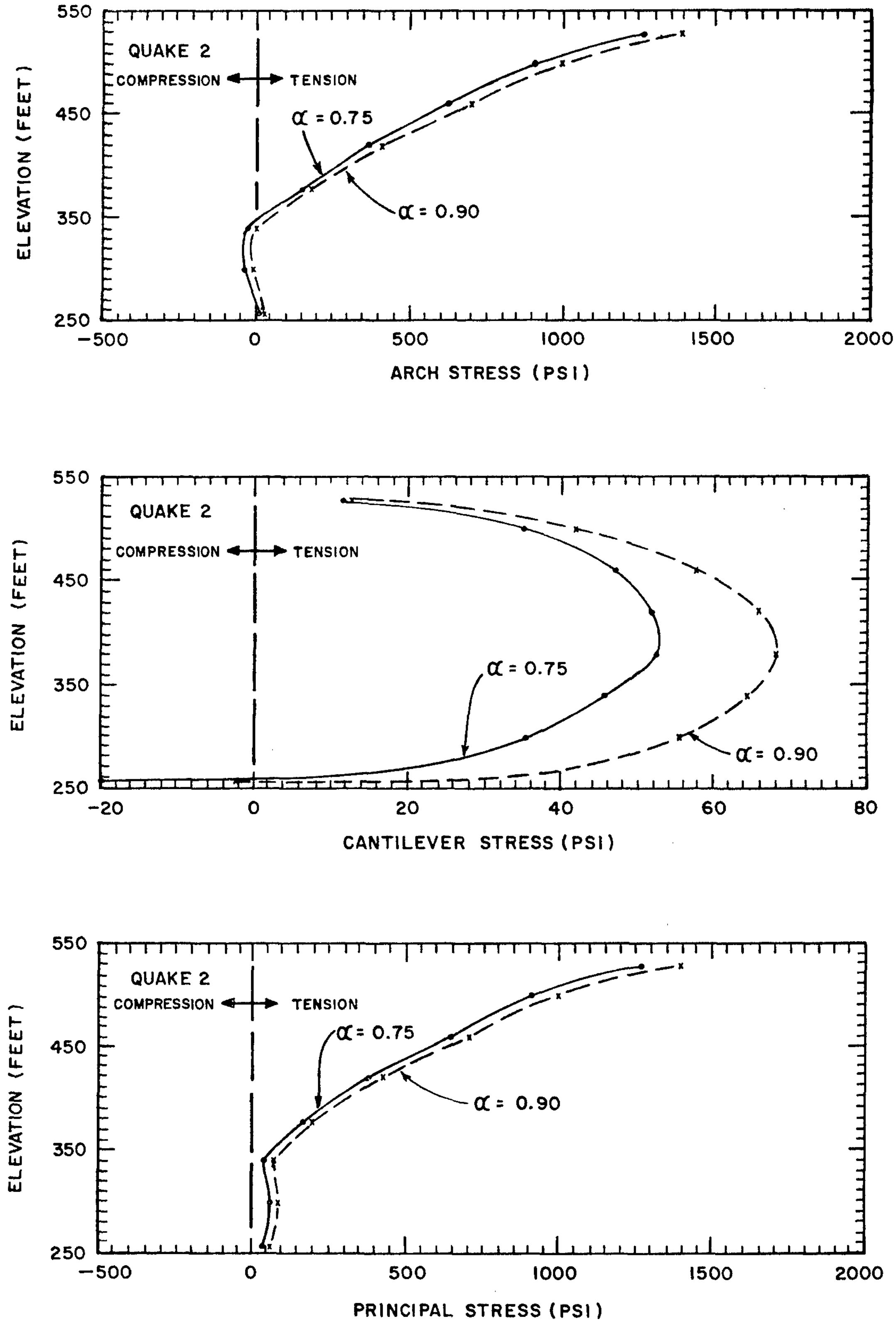


FIGURE E-32 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID-SURFACE OF THE DAM ALONG CROWN CANTILEVER. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$

STRESSES ON MID-SURFACE ALONG SECTION 8

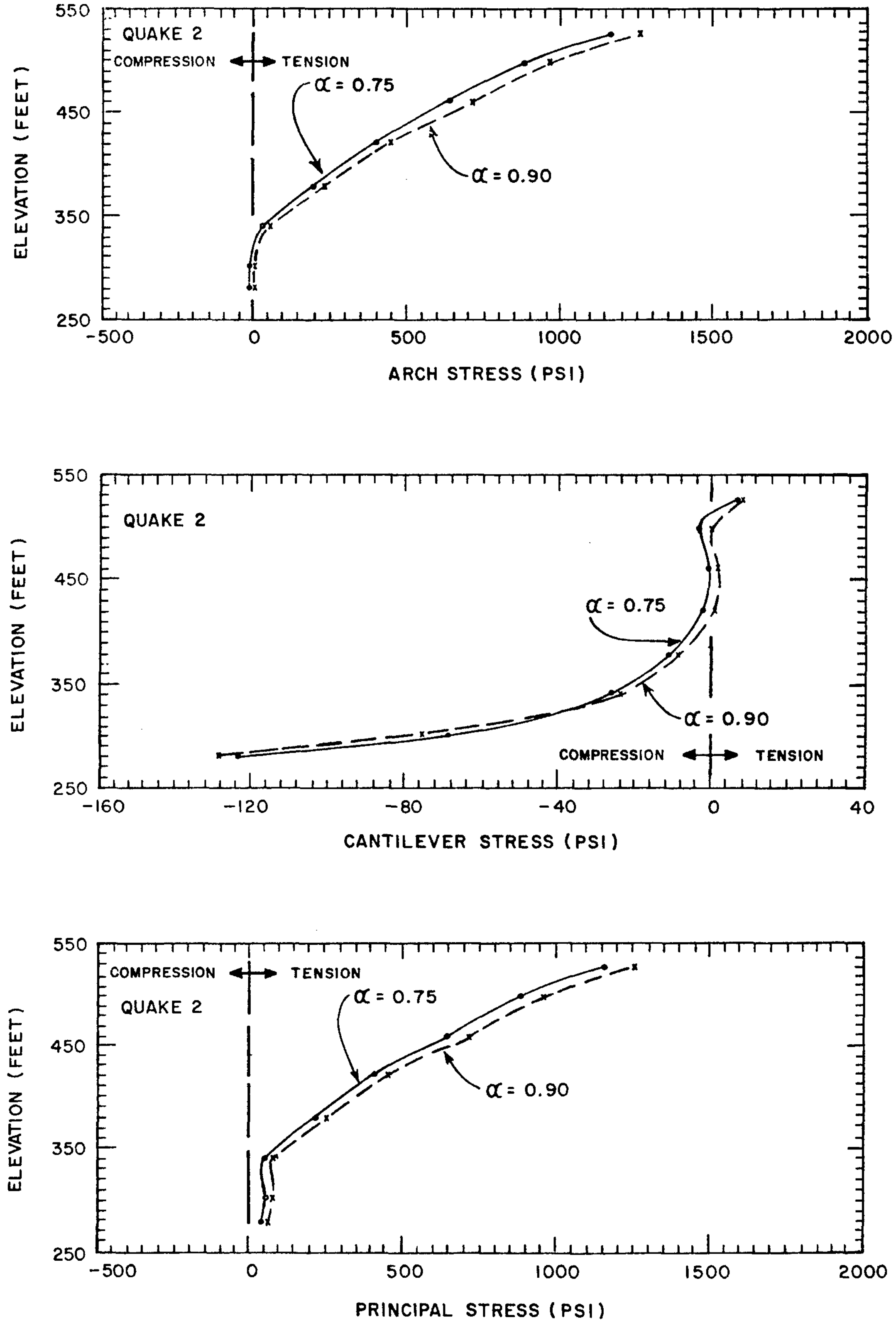


FIGURE E-33 PEAK STRESSES (ARCH, CANTILEVER, AND PRINCIPAL STRESSES) ON MID-SURFACE OF THE DAM ALONG SECTION 8. STRESS RESPONSE, COMPUTED FOR THE DAM ON FLEXIBLE FOUNDATION ROCK WITH FULL RESERVOIR (POOL ELEVATION = 527 FEET), IS DUE TO QUAKE 2 EXCITATION. STATIC STRESSES DUE TO DEAD WEIGHT OF THE DAM AND HYDROSTATIC PRESSURE ARE INCLUDED. $\eta = 0.14$